1st IRF Asia Regional Congress & Exhibition
November 17–19, 2014
Bali, Indonesia
Bali Nusa Dua Convention Center 2

International Road Federation
<table>
<thead>
<tr>
<th>TS 1.1 ASSESSING INFRASTRUCTURE PERFORMANCE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>• The most important aspects to select a measurement device for pavement bearing capacity survey</td>
<td>459</td>
</tr>
<tr>
<td>• Roadroid Continuous Road Condition Monitoring With Smart Phones</td>
<td>597</td>
</tr>
<tr>
<td>• Effective Maintenance Measures of Toll Road pavement by Private Company</td>
<td>772</td>
</tr>
<tr>
<td>• Development of Deduct Value Curves for Concrete Pavement based on Panel Rating Procedures</td>
<td>871</td>
</tr>
<tr>
<td>• A Robust In-Situ Displacement Measurement System of Bridge Structure by Using Digital Image Correlation Technique</td>
<td>940</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>TS 1.2 INFRASTRUCTURE MAINTENANCE POLICIES &amp; PROGRAMS</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>• Identification and Mitigation of Risks during Design and Implementation of Output and Performance based Road Contracts</td>
<td>279</td>
</tr>
<tr>
<td>• Developing a Trafficability Index of Vehicles during Winter</td>
<td>262</td>
</tr>
<tr>
<td>• The Study of Truck Transport Impacts on Rural Road Network for Future Road Maintenance Improvement Plan</td>
<td>588</td>
</tr>
<tr>
<td>• Pilot Surveys for Measuring Road Roughness Using a Smartphone in Papua New Guinea</td>
<td>848</td>
</tr>
<tr>
<td>• Asset Management to Drive Better Outcomes from Outsourced Road Maintenance and Renewal</td>
<td>880</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>TS 1.3 ROAD &amp; BRIDGE INVENTORY &amp; INSPECTION</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>• Long-term Bridge Maintenance and Management Plan for Rural Road in Thailand</td>
<td>8</td>
</tr>
<tr>
<td>• Collecting Pavement Big Data by using Smartphone</td>
<td>425</td>
</tr>
<tr>
<td>• WIM Bridge: Review And Future In Indonesia</td>
<td>765</td>
</tr>
<tr>
<td>• Getting Automatic Crack Detection Right for your Jurisdiction</td>
<td>724</td>
</tr>
<tr>
<td>• Road Project Delivery Method Selection Model: A Review for Indonesian Road Development</td>
<td>828</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>TS 2.1 PAVEMENT MANAGEMENT &amp; PRESERVATION</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>• The Role of Condition Benchmarking in Asset Management, Case Study for Pavement Asset in Abu Dhabi – UAE</td>
<td>417</td>
</tr>
<tr>
<td>• The Prediction Of Pavement Friction From Texture – Pitfalls And Potential</td>
<td>504</td>
</tr>
<tr>
<td>• Engineering Benefits of Pavement Management System Applications to Korea National Highways</td>
<td>671</td>
</tr>
<tr>
<td>• The Improvement of Pavement and Its Future Prospects in Taiwan</td>
<td>697</td>
</tr>
<tr>
<td>• Road Asset Management in Papua New Guinea</td>
<td>934</td>
</tr>
</tbody>
</table>
TS 2.3 DURABLE PAVING MATERIALS 1

- Improving Rutting Resistance and Moisture Susceptibility of Asphalt Binder and Mixtures Using Newly Developed Polymer-Modified Warm-Mix Asphalt Additive in Indonesia
- Utilization Of Asbuton As An Anti-Stripping Agent Of Asphalt Pavement
- Some concern for rational use of hydraulic graded iron and steel slag as reinforced base-course in Japan
- Laboratory investigation of asphalt binder with and without crumb rubber modifier
- Fiber Reinforced Asphalt Concrete: Performance Tests and Pavement Design Consideration

TS 2.4 DURABLE PAVING MATERIALS 2

- Pavement Aging Properties of Rubberized Asphalt and Neat Asphalt
- The Use of Ceramic Waste Materials as fine Aggregates in Hot Mix Asphalt
- Rubberized Asphalt Open Graded Friction Course History and Worldwide Use
- Study On Better Utilization Of Natural Rock Asphalt (ASBUTON) Between Indonesia and Japan
- Development of Warm-Mix Asphalt Technology Applied for Various Types of Asphalt Pavement in Korea

TS 2.5 DURABLE PAVING MATERIALS 3

- Material Characteristics of Polypropylene Coated Multi-filament Glass Fiber Reinforced Hot-Mix-Asphalt Mixtures
- Flow Number Properties of Stone Matrix Asphalt in Indonesia
- Performance-based Design Hot Asphalt Mix and Flexible Pavement – The European Perspective
- Pavement Distress Caused by Bitumen Hardening and Methods to Overcome
- Evaluation of Moisture Susceptibility of Asphalt Mixture Using Image Analysis and Performance tests

TS 2.6 DURABLE PAVING MATERIALS 4

- The Merits of Semi-Rigid Pavement and its Environmental Characteristics
- Laboratory Experimentation of Bituminous Foam Mix under Humid Curing Condition
- The mechanical and thermal analysis of porous asphalt concrete containing steel slags
- Continuously Reinforced Concrete Pavement (CRCP) Overlay Construction As A Solution For Concrete Pavement Deterioration Rehabilitation
- Warm Mix Asphalt for heavy Traffic in Indonesia
### TS 3.1 TRANSPORTATION FINANCING & ECONOMICS

<table>
<thead>
<tr>
<th>Topic</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>A Study on the Improvement of Private-Funded Expressways Operated By A Public Corporation</td>
<td>190</td>
</tr>
<tr>
<td>Establishing Optimal Long Term Funding Allocation Systematic Approach based on Network Needs &amp; Availability of Funds</td>
<td>199</td>
</tr>
<tr>
<td>The Unfinished Policy On Road User Charges And Road Preservation Unit: Indonesia's Homework to Implement Road User Charges</td>
<td>468</td>
</tr>
<tr>
<td>Analyzing Toll Road Service Quality From A Road User Perspective (Case Study of Toll Roads in Java)</td>
<td>627</td>
</tr>
<tr>
<td>Introduction and Effects of Environmental Road Pricing Scheme Where Inducing the Traffic Of Urban Expressway To Parallel Route by Discounting the Toll Fare</td>
<td>1026</td>
</tr>
</tbody>
</table>

### TS 4.1 INNOVATION IN ROAD PLANNING & CONSTRUCTION

<table>
<thead>
<tr>
<th>Topic</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Summary of Construction of Shorenji River Work Section on Yodogawa Sagan Route of Hanshin Expressway</td>
<td>645</td>
</tr>
<tr>
<td>The Impact of Road Network Development on Land Use: Case Study Karawang Regency</td>
<td>689</td>
</tr>
<tr>
<td>Monitoring of Structural Behavior of Corrugated Steel Plate Underpass during Construction</td>
<td>719</td>
</tr>
<tr>
<td>Development of Unit Weight Based Technique For Verification of Water: Binder Ratio Of Field Concrete</td>
<td>746</td>
</tr>
<tr>
<td>Segmental-Orthotropic-Steel Panel Behaviour on Citarum 1 Bridge, Bandung Regency</td>
<td>1018</td>
</tr>
</tbody>
</table>

### TS 4.2 INNOVATION IN BRIDGE CONSTRUCTION

<table>
<thead>
<tr>
<th>Topic</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Institutional Approach In Strengthening Indonesia Bridge Management: A Lesson Learned From Collapsed Kutai Kartanegara Suspension Bridge</td>
<td>104</td>
</tr>
<tr>
<td>A Study on Risk Evaluation Method for Bridge Asset Management</td>
<td>238</td>
</tr>
<tr>
<td>The use of fiber concrete for bridge construction</td>
<td>271</td>
</tr>
<tr>
<td>A Review on Indonesia’s Highway Bridge Construction Specification In Order To Support Trans-Asian Highway</td>
<td>300</td>
</tr>
<tr>
<td>Construction of an Expressway Bridge having Butterfly–Shaped Web</td>
<td>916</td>
</tr>
</tbody>
</table>

### TS 5.1 CRASH DATA ANALYSIS & REPRESENTATION

<table>
<thead>
<tr>
<th>Topic</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Examining Factors Affecting the Severity of Run-off-Road Crashes: Abu Dhabi Case Study</td>
<td>168</td>
</tr>
<tr>
<td>Characterization of Pedestrian Fatalities in Urban Arterial Corridor in Puerto Rico</td>
<td>364</td>
</tr>
<tr>
<td>Ordered Logit Model for Severity analysis of the accident on Thailand Rural Road Network</td>
<td>477</td>
</tr>
<tr>
<td>The Development of Motorcycle Crashes Prediction Model on Collector Roads By Using Generalized Linear Models</td>
<td>572</td>
</tr>
<tr>
<td>Roadway Safety in Eastern Province of Saudi Arabia: Crash Data Evaluation and Spatial Analysis</td>
<td>788</td>
</tr>
</tbody>
</table>
TS 5.2 DESIGNING SAFER ROAD SIDES

- Three-Cable Barrier System Adjacent to Steep Slopes 71
- Functional Limits Of The W-Beam Guardrail 206
- The Fatal and Serious Injury Risk of Motorcycle Collisions with Traffic Barriers 333
- A MASH Compliant Sign Mounting Designs for Placement on Concrete Median Barrier 341
- Optimizing Crash Cushion Selection based on Performance, Physical Constraints and Reusability 814

TS 5.3 URBAN SAFETY TREATMENTS

- Characteristics of serious crashes at signalized intersections In Abu Dhabi City, UAE 17
- Case Study of Urban Acupuncture: Bicycle Lane in Bandung City 42
- Accessibility and Mobility Improvement Through Skywalk And The Arrangement Of Pedestrian Network System 50
- Modern Roundabout Safety Assessment in the United States 797
- Analysis of Spatial Patterns and Influence Factors of Urban Traffic Accidents: A Case of Seoul, Korea 144

TS 5.4 SPEED & TRAFFIC ENFORCEMENT

- Integrated Road Safety Management in Indonesia and The Role of the Indonesian National Traffic Police Corps 57
- A Study on Media Exposure among Malaysian Road Users for Effective Communication on Speed Cameras Implementation 564
- The Relationship Between the Use of Traffic Safety Technologies & the Drivers Behavior in Abu Dhabi Highways 580
- Effective Automated enforcement 608

TS 5.5 NATIONAL SAFETY PROGRAMS

- Road Safety Assessment of Southern East Java National Corridor 118
- Road Safety Policies in Indonesia - The Decade of Action for Road Safety 2011-2020 520
- Vaccines for roads: road assessment program initiatives in India and China 804
- Road Traffic Accidents and Fatality Rates in Libya 757
- IRF Driver Behavior Education & Training Sub-Committee (DBET) Position Statement & Guidelines 960
**TS 5.6 SAFETY TREATMENTS**

<table>
<thead>
<tr>
<th>Treatment</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>A Speed Restraint Pavement with a Longitudinal Surface Profile of Sine Waves</td>
<td>945</td>
</tr>
<tr>
<td>Selection of Road Safety Measures According to Capacity, Safety and Cost Approach</td>
<td>496</td>
</tr>
<tr>
<td>Cost-Effective Safety Treatment of Culverts and Bridges on Low-Volume Rural Roads</td>
<td>654</td>
</tr>
<tr>
<td>Cost-Effective Safety Treatment of Foreslopes and Ditches on Low-Volume Rural Roads</td>
<td>733</td>
</tr>
<tr>
<td>Quantifying Risk for Safer Roads Using ChinaRAP - Case Study of S102 Trunk Road in Shaanxi Mountain</td>
<td>888</td>
</tr>
<tr>
<td>Guaranteed Rumble Dot At Tangerang Merak Toll Roads</td>
<td>1011</td>
</tr>
</tbody>
</table>

**TS 5.7 SAFETY PERFORMANCE**

<table>
<thead>
<tr>
<th>Performance</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Guideline for Road Safety Countermeasures</td>
<td>215</td>
</tr>
<tr>
<td>Development of Safety Performance Functions for Freeways in Puerto Rico</td>
<td>374</td>
</tr>
<tr>
<td>Safety Performance Functions &amp; Safety Conscious Planning Indiana - A Case Study</td>
<td>780</td>
</tr>
<tr>
<td>Countermeasure for Speed Reduction Effect in Tunnel Section By Sequence Design Study Based on Driver's Sensation</td>
<td>952</td>
</tr>
<tr>
<td>The Minimum Safety Service Standard On Padalarang-Cileunyi Toll Road</td>
<td>995</td>
</tr>
</tbody>
</table>

**TS 6.1 ADVANCED INTELLIGENT TRANSPORT SYSTEMS**

<table>
<thead>
<tr>
<th>System</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Research on Advanced Road Management using “ITS Spot” in Japan</td>
<td>135</td>
</tr>
<tr>
<td>UIRNet: The Italian National ITS Platform for integrated logistics</td>
<td>222</td>
</tr>
<tr>
<td>The IRF Vienna Manifesto on ITS: Smart Transport Policies for Sustainable Mobility</td>
<td>904</td>
</tr>
</tbody>
</table>

**TS 6.2 INTELLIGENT INFRASTRUCTURE**

<table>
<thead>
<tr>
<th>Infrastructure</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Construction and Operation of SMART Highway Test Bed</td>
<td>128</td>
</tr>
<tr>
<td>Traffic Flow Analysis by the Use of Wi-Fi Packets Receiver</td>
<td>680</td>
</tr>
<tr>
<td>Evaluation of the Radar Detector Developed as The Next Generation Automatic Incidents Detector in Korea</td>
<td>820</td>
</tr>
</tbody>
</table>
### TS 6.3 MANAGING URBAN MOBILITY

<table>
<thead>
<tr>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>The Use of Speed Limit As Congestion Threshold for Performance Measures</td>
<td>356</td>
</tr>
<tr>
<td>Mitigating Traffic Congestion in Asia: A comparative Analysis</td>
<td>451</td>
</tr>
<tr>
<td>Minimizing Congestion on the Trans-Asian Highway</td>
<td>538</td>
</tr>
<tr>
<td>Variable Message Signs As A Solution toO Overcome Congestion In Urban Road</td>
<td>619</td>
</tr>
<tr>
<td>Public Spaces in Historic Cities and Vulnerable Users Movement</td>
<td>1003</td>
</tr>
<tr>
<td>The Impact of High Occupancy Vehicle Policy on Traffic Performance of Dr. Djundjunan Street In Bandung Indonesia</td>
<td>984</td>
</tr>
</tbody>
</table>

### TS 7.1 SUSTAINABLE TRANSPORT PRACTICES

<table>
<thead>
<tr>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>The Plans to Support the Operations of Express Buses via JOBAN Expressway as countermeasures Against Great East Japan Earthquake</td>
<td>230</td>
</tr>
<tr>
<td>Mode Separation with a Purpose – A Traffic Management Approach to Bring Order in Dhaka’s Chronic Traffic Problems</td>
<td>247</td>
</tr>
<tr>
<td>Benefit Evaluation of Road Rehabilitation at Nine Provinces in Indonesia</td>
<td>288</td>
</tr>
<tr>
<td>Improve the Regional Accessibility Through Road Network Development in the Border Region of Indonesia</td>
<td>383</td>
</tr>
<tr>
<td>The Accessibility of Paloh-Aruk Border Area at Sambas Regency West Borneo</td>
<td>442</td>
</tr>
<tr>
<td>A Comparative Analysis of Child-Friendly Transportation Between Canada and Indonesia</td>
<td>547</td>
</tr>
</tbody>
</table>

### TS 7.2 TRANSPORT, ENERGY & CLIMATE CHANGE

<table>
<thead>
<tr>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>CO2 Impact of 2 Wheelers In Asian Countries</td>
<td>182</td>
</tr>
<tr>
<td>Implementation of Thermoelectric effect to Road Facilities</td>
<td>256</td>
</tr>
<tr>
<td>Nanotechnology for Green Roads</td>
<td>311</td>
</tr>
<tr>
<td>Joint Research on Carbon Dioxide (CO₂) Emission from Motorcycle between Indonesia and Japan</td>
<td>529</td>
</tr>
</tbody>
</table>
Long-term Bridge Maintenance and Management Plan for Rural Road in Thailand

Saiprasertkit Kawin, Wittitanapanit Jarurat and Bamrungwong Chakree
Department of Rural Road, Ministry of Transportation, Thailand

Email for correspondence: ss.kawin@gmail.com

1. INTRODUCTION

Department of Rural Road, a government body operating under the Ministry of Transport, in charge of maintaining and developing the state road by considering both engineering and social needs. In general, the department monitors 47,734km of collector roads around the country, consisting of 42,634km of paved roads and 5,100km of laterite roads. The department is also managing more than 8,000 numbers of bridges [1]. Currently, most of them are already damaged or being damaged due to lack of repair so that they are increasingly failing to give the appropriate road network service. In the past, to maintain the services, minimum budget was allocated annually, approximately 700 us dollars per bridge. However, this seems not enough to fully cover the cost of maintenance operation. Aside from lack of repair, flood disaster may often take place during rainy season coming every year and threaten road network function. Hence, bridge maintenance and management is essential for sustaining the service as well as preventing it from damages caused by natural disaster.

Long-term maintenance planning is intended to provide plans concerning preventive repairs and planned rebuilding with the aims of extending the service life of bridges and reducing bridge maintenance costs as shown in Figure1.

![Diagram](image)

**Figure1** Long-term maintenance plan flow and examination items.
2. MAINTENANCE SCENARIOS BASED ON CONTROL LEVEL

2.1 NECESSITY OF CONTROL LEVEL

Bridges managed by Department of Rural Roads differ widely in terms of size of bridge, traffic volume, location conditions (emergency transport route, intersection, etc.) and conditions of use. Implementing maintenance of such bridges based on the same standard is an inefficient approach. Therefore, the concept of control level is introduced in this project as a mean to effectively maintain all of bridges that are under Department of Rural Road management. In the concept of control level, each bridge will firstly be categorized into one of the four levels, namely A to D, corresponding to its level of importance. Then maintenance will be executed according to each control level repair plan. Through introducing the control level, it becomes possible to prepare the maintenance scenario for each bridge in respect to its importance degree.

The control level may be described as the maintenance goal, and this identifies a goal for “maintaining soundness of the bridge concerned at a certain level” and compiling a plan for attaining the goal. Setting maintenance scenarios based on the control level entails setting the timing of maintenance and repair measures for each bridge, which is linked to establishing the order of priority of maintenance and repair measures.

For example, in the case where there are numerous bridges that have the same extent of damage. The bridges with a high control level will be those that have reached or exceeded the scheduled maintenance and repair time, and they will be deemed to require immediate measures. Whereas the bridges with a low control level will be those that have not reached the scheduled maintenance and repair time, and it will be deemed permissible to leave them unattended for the time being. In the current situation where it is imagined there are numerous bridges with damage, through introducing the control level approach, it should be possible to rationally disperse the initial investment that is concentrated in the planning stage. The following sections outline each control level. Overall image of the aforementioned Control Levels A to D is shown in Figure 2. This figure shows bridge maintenance is performed after calculated soundness value dropped below certain point predetermined by which control level the bridge belongs to.

![Figure 2 Image of Control level.](image-url)
2.2 SETTING OF THE CONTROL LEVEL

It is necessary to set the control level of each bridge upon considering the social conditions, service conditions and environmental conditions and so on that the bridge is placed under. When setting the control level of bridges managed by Department of Rural Roads, a method that allows the bridges to be quantitatively evaluated according to indicators (importance evaluation indicators) that reflect the importance of bridges. Table 1 shows the importance evaluation indicators that are used in the project. The priority coefficient values in Table 1 and Priority evaluation point Eq. (1) are obtained from Jica Thailand.

Table 1 Priority evaluation index set-up

<table>
<thead>
<tr>
<th>Items</th>
<th>Priority coefficient</th>
<th>Limit and Score (max 10 and min 0)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roadway</td>
<td>3</td>
<td>0-10 scores depending on the degree of priority to roadway</td>
</tr>
<tr>
<td>Crossing state</td>
<td>3</td>
<td>0-10 points converted from calculation in other way</td>
</tr>
<tr>
<td>Total traffic</td>
<td>3</td>
<td>Railway: ≥ 20,000, Highway: ≥ 10,000, Local road: ≥ 3,000, Dam: ≥ 2,000, Others: &lt; 500</td>
</tr>
<tr>
<td>Heavy vehicle traffic</td>
<td>2</td>
<td>Railway: ≥ 4,000, Highway: ≥ 2,000, Local road: ≥ 1,000, Others: &lt; 1,000</td>
</tr>
<tr>
<td>Bridge length</td>
<td>1</td>
<td>Bridge length: Proportional distribution (0-10)</td>
</tr>
</tbody>
</table>

As mentioned in the introduction, roadway, crossing state and bridge length are representative for social factor, while, total traffic and heavy vehicle traffic are considered as engineering factor. The value calculated from the importance evaluation shall be called the “importance evaluation score.” The importance evaluation score is calculated through multiplying the importance coefficient by the score for each indicator and deriving the total for each item as indicated in Eq. (1).

\[
\text{Priority evaluation point} = \sum (\text{priority coefficient} \times \text{score}) \quad \text{Eq. (1)}
\]

3 SOUNDNESS EVALUATION

Each bridge will be periodically evaluated so that Department of Rural Roads is always aware of their conditions by using method of soundness evaluation. Decision for maintenance execution is also based on the level of damage indicated by soundness evaluation.

3.1. DEFINITIONS OF DAMAGE CLASSIFICATION AND SOUNDNESS

In order to compile the long-term maintenance plan, it is important to quantitatively gauge the condition of bridges. Here, as the method for doing this, the soundness of bridges (members) shall be calculated. Degree of damage and soundness are defined as follows.

Damage classification: This is an indicator of the level of seriousness of damage in members that are the units of inspection indicated in the “Inspection Work and Evaluation Manual [2].” The damage classification is expressed by the results of damage judgment obtained from inspection.

Soundness: This indicator expresses the functional maintenance level of members or bridges. In another way, it is an indicator for gauging the overall condition of members or bridges upon considering the fluctuation or range of damage classification confirmed for each member number.
3.2 SELECTION OF MEMBERS FOR SOUNDNESS EVALUATION

Bridges are composed of numerous members, and the degree of impact on bridge structural safety differs according to each member as shown in Figure 3. Accordingly, members are classified into main members and other members according to the importance. Here, main members are defined as members which if left unattended may cause necessity to rebuild the bridge. Therefore, the evaluation of soundness for compiling the long-term maintenance plan in Department of Rural Roads will target on main girders, crossbeams, slab decks, bearings, abutments and bridge piers. The damage evaluation classification for key members are indicated in Table 2. The meaning of 1 is healthy condition while 5 is severe damage condition.

Table 2 Damage evaluation classification and damage type in inspection work and evaluation manual

<table>
<thead>
<tr>
<th>No.</th>
<th>Damage</th>
<th>Evaluation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Defect in PC anchorage detail</td>
<td>1, 5</td>
</tr>
<tr>
<td>2</td>
<td>Crack, leakage, freeline</td>
<td>1, 2, 3, 4, 5</td>
</tr>
<tr>
<td>3</td>
<td>Concrete slab deck falling out</td>
<td>1, 3, 5</td>
</tr>
<tr>
<td>4</td>
<td>Crack on slab deck</td>
<td>1, 2, 3, 4, 5</td>
</tr>
<tr>
<td>5</td>
<td>Rebar exposure</td>
<td>1, 3, 5</td>
</tr>
<tr>
<td>6</td>
<td>Bearing malfunction</td>
<td>1, 3, 5</td>
</tr>
</tbody>
</table>

3.3 SOUNDNESS CALCULATION

The soundness is expressed by a score out of 100 points. The entirely sound state where there is no damage at all (for example, immediately after completion of the bridge) is given a score of 100, while the state where the impacts of damage hinder traffic and make it necessary to conduct load restrictions and traffic controls, etc. is given a score of 0. The overall state of damage of members is numerically expressed as the “Overall degree of damage,” and the soundness of members is calculated according to the following formula Eq. (2).

\[
\text{Soundness} = 100 - \text{Overall degree of damage}
\]  

Eq. (2)

The damage evaluation classification of each element is numerically defined as a damage score in Table 3, and the overall degree of damage is calculated in consideration of the ratio of number of damaged members out of the total number of member numbers. Moreover, concerning the damage evaluation classification of each member number for which soundness is being calculated, data on inspection results will be used according to the Inspection Work and Evaluation Manual [2].

Table 3 Damage evaluation classification in inspection work and damage score
### Table 3

<table>
<thead>
<tr>
<th>Damage evaluation</th>
<th>Damage</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>No damage</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>20</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>50</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>70</td>
</tr>
<tr>
<td>5</td>
<td>Serious damage</td>
<td>90</td>
</tr>
<tr>
<td>U (E, R)</td>
<td>Urgent care is required</td>
<td>200</td>
</tr>
</tbody>
</table>

The damage evaluation classifications prescribed in the Inspection Work and Evaluation Manual [2] are stipulated as 1~5 and E and R as shown in Table 3, however, in order to conduct the future simulation, it is necessary to consider the interference to traffic if the 5 classifications are left unattended. Therefore, it has been decided to add a new classification of U (= Urgent) as the next level of damage after the five classifications. Moreover, the U classification is only a future simulation setting, i.e. it is an evaluation classification that isn’t inputted as a damage classification in inspections.

\[
D = 20 \times D_1 + 50 \times D_2 + 70 \times D_3 + 90 \times D_4 + 200 \times D_5
\]

Where, \( D \) : Overall degree of damage.

\( D_1 \) - \( D_5 \) : Ratio of member numbers of evaluation classification 2,3,4,5 and U respectively.

Since Evaluation classification U, which is separately set for conducting simulation of future conditions, targets damage that is serious enough to impede traffic safety, its damage score is set at 200 in consideration of its importance and in order to thoroughly ensure that risk is averted.

### 3.4 EXAMPLE OF SOUNDNESS CALCULATION

Overall degree of damage of Figure 4: \( D = 20 \times 0.50 + 50 \times 0.25 + 70 \times 0.25 + 90 \times 0.0 + 200 \times 0.0 = 40.0 \). Therefore, slab deck soundness = 100 - 40.0 = 60.0. The meaning of rate is ratio between number of members with same type of damage and total members.

### Figure 4 Inspection result of slab deck damage

In cases of damage classed as 5, which is the most serious state of damage, in the periodic inspection, the results of inspection are reevaluated by expert engineers from the viewpoint of risk aversion. As a result of the re-evaluation, the damage classification 5 is classified into three types as shown in Table 4.

### Table 4 Damage classification

<table>
<thead>
<tr>
<th>Classification</th>
<th>Evaluation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Emergency repair is needed to recover damage in short time</td>
<td>E</td>
</tr>
<tr>
<td>2. maintenance and repair for damage with high priority</td>
<td>R</td>
</tr>
<tr>
<td>3. Other damages excluded from 1,2</td>
<td>5</td>
</tr>
</tbody>
</table>

Classification 1 refers to damage that requires urgent attention, and Classification 2 refers to damage with a high priority for maintenance and repair. Evaluation methods in the long-term maintenance plan are indicated below.
(1) Classification 1: damage that requires urgent attention (E)

Since traffic is already hindered due to bridge collapse and so on, urgent attention is needed. Accordingly, in the long-term maintenance plan, this is not targeted for evaluation of soundness, and the following handling is conducted in the simulation: Members that are evaluated as “E” following re-evaluation of the inspection results are omitted from simulation. Assuming that the members evaluated as “E” undergo complete maintenance and repair countermeasures under urgent response from the day of inspection implementation, they will be reverted to simulation from the fourth year onwards. Soundness at the time of reversion will be 100 points.

(2) Classification 2: damage with high priority for maintenance and repair (R)

Within damage classification 5, this refers to damage that is deemed to have particularly high urgency and to carry a risk of critically impacting bridge safety by expert engineers. Accordingly, the soundness of members experiencing damage having a high maintenance and repair priority (R) is calculated. Moreover, if damage having a high maintenance and repair priority (R) occurs in even one member number, the soundness of members is evaluated as 10.

4 METHOD FOR FUTURE PREDICTION OF SOUNDNESS

In order to compile a preventive maintenance plan for taking measures that are based on the medium- to long-term viewpoint, it is necessary to determine the future state of degradation of bridges. Markov’s transition probability [3, 4] is adopted. The Markov’s transition probability is a soundness calculation method for gauging the condition of bridges in which all inspection data (degree of damage to each member number) are used to evaluate the degree of soundness of members overall upon considering the scale, scope and variance of damage. Through using this method, it is possible to effectively utilize the bridge inspection data that will be accumulated from now, appropriately evaluate the scope and variance of damage based on the weighted mean of all inspection data, and improve the accuracy of budget planning.

4.1 MARKOV’S TRANSITION PROBABILITY

Markov’s transition probability is a model for indicating the probability that one state of being will change to the next state of being. For example, assuming that only elements with probability of P_x will move from degradation state 0 to degradation state 1 in a year, the remaining elements (1-P_x) will remain at degradation state 0 as shown in Figure 5. By conducting this repetitive calculation once a year, it is possible to calculate the distribution of probability indicating the state of degradation.

\[
\begin{array}{cccccc}
\text{Degradation degree} & 0 & 1 & 2 & 3 & 4 & 5 \\
\text{Transition probability, p_x} & 1-p_x & 1-p_x & 1-p_x & 1-p_x & 1-p_x & 1-p_x \\
\end{array}
\]

\[
\begin{array}{cccccc}
\text{From} & 1 & 2 & 3 & 4 & 5 & U \\
1 & 0.900 & 0.100 & 0.000 & 0.000 & 0.000 & 0.000 \\
2 & 0.000 & 0.900 & 0.100 & 0.000 & 0.000 & 0.000 \\
3 & 0.000 & 0.000 & 0.900 & 0.100 & 0.000 & 0.000 \\
4 & 0.000 & 0.000 & 0.000 & 0.900 & 0.100 & 0.000 \\
5 & 0.000 & 0.000 & 0.000 & 0.000 & 0.900 & 0.100 \\
U & 0.000 & 0.000 & 0.000 & 0.000 & 0.000 & 1.000 \\
\end{array}
\]

Figure 5 Example of Markov’s transition probability matrix

4.2 PREDICTION USING MARKOV’S TRANSITION PROBABILITY

Markov’s transition probability matrix is assumed to be as indicated in Figure 5. The above table shows the transition probability matrix where 10% of the member numbers that are evaluated with an “1”
classification this year move to the “2” classification next year, while the remaining 90% of member numbers remain in the “1” category.

5. SETING OF MAINTENANCE AND REPAIR MEASURES ACCORDING TO SOUNDNESS

Relation between the Definition of Measures and Control level is considered. Table 5 shows the relationship between the level of countermeasures and soundness of member.

<table>
<thead>
<tr>
<th>Control level</th>
<th>Definition of repair</th>
<th>Before-repair soundness</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Preventive action</td>
<td>80</td>
</tr>
<tr>
<td>B</td>
<td>Repair 1 (slight damage)</td>
<td>60</td>
</tr>
<tr>
<td>C</td>
<td>Repair 2 (medium damage)</td>
<td>40</td>
</tr>
<tr>
<td>D</td>
<td>Repair 3 (serious damage)</td>
<td>20</td>
</tr>
<tr>
<td>Other</td>
<td>Bridge replacement</td>
<td>0</td>
</tr>
</tbody>
</table>

6. PRIORITY OF COUNTERMEASURES

In order to compile the long-term maintenance plan, it is necessary to calculate the feasible single year budget for DRR while sustaining the future soundness of bridges (i.e. equalize the budget), and in order to consider budget equalization it is necessary to decide the order of priority of maintenance. Figure 6 sums up the previously mentioned approaches used when deciding the order of priority of maintenance. As shown in the Figure 6, step 1 is annual project cost for all expenses. Step 2 is project cost breakdowns and Step 3 is plan including priority degree. The expenses are allocated within annual budget per year. For the suspended bridge, the degradation process is considered so that project cost is increased.

The method for determining the order of priority of countermeasures in Department of Rural Roads is indicated as it is aimed at securing future bridge soundness and reducing the maintenance budget while giving top priority to securing safety. Level of priority can be explained as follows.
1\textsuperscript{st} Bridges with soundness of less than 10: Top priority shall be given to bridges with soundness of less than 10 because there is a risk of bridge collapse or impairment of traffic safety. It is essential to give top priority to risk management and ensure that there are no bridges that end up rebuilding.

2\textsuperscript{nd} Bridges with a major drop in the soundness: As the next order of priority, measures will be conducted on those bridges that have a large drop in soundness judging from the timing of countermeasures.

3\textsuperscript{rd} Bridges with a low control level: In cases where the drop in soundness is the same value, maintenance work will first be conducted from bridges with a low control level.

4\textsuperscript{th} If the control level is the same, bridges that have a higher importance evaluation score: In cases where the drop in soundness and the control level are the same, priority shall be given to bridges that have a higher importance evaluation score. An example of calculating the order of priority is indicated in Figure 7.

7. CONCLUSIONS

The introduction of Long-term Bridge Maintenance and Management Plan will enable Department of Rural Roads to manage bridges effectively. Periodic inspection, which is part of the maintenance plan, will also give updated conditions of supervised bridges, so that Department of Rural Roads can grasp the current situation of each bridge. More importantly, by implementing Long-term Bridge Maintenance and Management Plan, it is possible to control the budget during the fiscal year and avoid unexpected expense due to collapse of bridges. However, due to lack of repair and inspection for long time, it is difficult to directly apply the simulation results as the countermeasure refers to member damages. Thus, in the initial stage, the countermeasure will prioritize to overall bridge damages. Then, after most of bridges with serious damages are repaired, the countermeasure for each member damage will be considered.

REFERENCES


Characteristics of serious crashes at signalized intersections
In Abu Dhabi City, UAE

Corresponding Author:

Mohammed Kishta, M.A.SC
Road and Traffic Engineer
Traffic and Patrols Directorate
Abu Dhabi Police
Abu Dhabi, UAE
+971503220883
eng.m.kishta@gmail.com

Atef Garib, Ph.D.
Road and Traffic Expert
Traffic and Patrols Directorate
Abu Dhabi Police
Abu Dhabi, UAE
+97125127176
agarib@eim.ae

Hussain Al-Harthei, M.A.SC
Director
Traffic and Patrols Directorate
Abu Dhabi Police
Abu Dhabi, UAE
+97125127222
alharthei@saaed.ae

Paper submitted for presentation at the 1st IRF Asia Regional Congress,

July 30th, 2014
ABSTRACT
Traffic signals are mainly implemented to completely avoid or reduce the number of traffic conflict points at intersections. However, in practice many traffic crashes occur at signalized intersections. Understanding the main causes of such crashes is an important approach to improve traffic safety at signalized intersections.
This study aims to investigate the factors that affect the traffic crashes type and severity at Abu Dhabi City signalized intersection. The studied factors include location of intersection (at CBD “Central Business Districts” / out of CBD), phasing sequence (split / lead-lag), time of day, day of week, etc..
The main objectives of this study are to assess the current traffic safety performances of Abu Dhabi signalized intersections.

Preliminary results indicated that 843 crashes been in Abu Dhabi signalized Intersections between 2008 and 2013, 35.8% of them been on CBD. The total number of injuries and fatalities were 1635. This paper will present detailed data collection and data analysis efforts, Also it will highlight main findings. The conclusion and recommendations will be used to improve traffic safety plans in Abu Dhabi.

INTRODUCTION
Traffic safety concept aims to adopt all plans, programs, regulations and preventive measures to reduce or prevent the occurrence of traffic accidents in order to ensure human safety, property and to preserve the security of the country and its components human and economic. So traffic safety became one of the basic elements of public health, which attaches a lot of countries special attention and considers social responsibility must be supported by all segments of society, including government and private institutions, NGOs, charities and individuals. Most of both developed and developing countries maintaining road users safety by numerous traffic control, perhaps the most important signalized intersections.
The main objectives of traffic control devices are to improve road users’ safety and advance traffic flow. When we are talking about signalization we can conclude that the major objective of Traffic Signal design is to maintain the free flow of traffic by assigning best green time to vehicle movements (signal phases).
Signalized intersections considered as the most critical locations of a roadway network due to traffic conflicting movements and frequently changing traffic signals. (F. Guo et al. 2010) mentioned that in their report on 2005, FHWA concluded that in the United States (year 2000), there were more than 46% signalized intersections crashes of intersection-related crashes. That crashes left 445,000 injuries.
(Rice, 2007) concluded that crashes related to signalized intersections tend to be more severe. He found that 30% of intersection-related fatalities occurred at signalized intersections while only 10% of intersections are signalized. So, improving intersection safety has been considered as a top priority by U.S.A. federal, state, and local agencies (FDOT, 2012).
This study aimed to identify the important driver, vehicle, nature, timing, and crash related factors that influence the severity of signalized intersections’ crashes. This will
help to develop solutions and mitigations that will contribute for improving efficiency of the traffic and raise traffic safety performances.

Also a logistic regression analysis will apply to model the impact of the studied factors on the crash type and severity. Logistic regression analysis is used to investigate the association between a number of explanatory variables and a single response variable—crash severity. The factors identified in this study are expected to help developing potential countermeasures, which will ultimately reduce the severity as well as the number of signalized intersection crashes.

LITERATURE REVIEW

Signalized intersections’ crash is one of the most dangerous phenomena on traffic safety in any country, and it is one of the main reasons that often result in fatalities and serious injuries. As known, there is a big problem related to traffic crashes at signalized intersections, which they threaten pedestrian who cross the road without looking in the direction of vehicles for their confidence that no one will cut off the signal.

According to the NHTSA’s National Center for Statistics and Analysis (Traffic Safety facts, 2004 data), 42,636 people died in motor vehicle crashes in 2004 alone and an additional 2,788,000 people were injured. There were over 6 million police reported auto accidents in 2004. It is reported that about 50% of crashes occur at the intersections (Sayed et al. 1999).

One main goal of intersection safety studies is to identify high risk factors among intersection geometric design features, traffic control and operational features, and traffic flow characteristics. (Wang et al. 2006) have shown that the traffic volume per lane has a significant impact on safety. Furthermore, there is a significant association between through/left-turning movements and the rear end, right-angle, and left-turn crashes (Wang and Abdel-Aty, 2007, 2008). Intersection geometric design features (i.e., number of through lanes, right-turn lanes, left-turn lanes, etc.) and traffic control and operational features (i.e., signal phase, speed limit, etc.) were also found to have significant influence on crash occurrence (Abdel-Aty and Wang, 2006; Wang et al., 2008). Wang et al. (2006) mentioned the following conclusions reached by examining the model output.

1. Traffic conditions as measured by the standardized ADT per lane by turning movement on the major and minor roads have a significant impact on the safety of signalized intersections.
2. Intersection size is closely related to intersection safety. In general, larger intersections are more dangerous than smaller intersections.
3. Signal coordination shows negative impact on safety.

In a research conducted in the City of Norfolk for signalized intersections on 2010 by (Sharad K. Maheshwari, Kelwyn A. D’Souza) to study the signalized intersection in a city to delineate intersection geometry and design factors which may be contributing to traffic accidents, a linear regression model was used to establish relationship between these variables and the accident rate. The resulting regression model explained 60% of the variability. It also showed that four topographical variables are more important than
other variables. These variables include number of lanes, number of turn lanes, presence of median and presence of permanent hazard like railway crossing.

A study by Corben and Foong, 1990 led to development of a seven-variable linear regression model for predicting right-turn crashes at signalized intersections. This model explained 85% of the variance of accident occurrence.

In a study conducted in the City of Victoria, Australia, for crashes occurred at signalized intersections on 1994 by (K.W. Ogden et al. 1994) they concluded that factors other than traffic volumes are contributing to accidents at signalized intersections. An important conclusion of that study is that there is some evidence to suggest that accidents at signalized intersections are to an extent affected by driver visual misperception of the presence of an intersection or the appropriate response to it. That study also found that there is some tendency towards higher accident frequencies at sites with narrower lanes. Therefore, lanes of less than normal width should be avoided where possible.

(Hussein A. Ewadh, Sahar S. Neham, 2007) conducted such study at four leg-signalized intersections using traffic conflict technique. Developed models show that, linear relationships are significantly appropriate to explain the relation between hourly traffic conflict and hourly approach traffic volume with coefficient of determination ranging between 0.701 and 0.884. Although this finding agrees with the literatures, exponential relationship introduces higher coefficient of determination between 0.759 and 0.897. This might be due to extra increase in conflicts at high traffic volume.

(Sharad K. Maheshwari, Kelwyn A. D’Souza, 2008) conducted pilot study to apply a proactive approach using traffic pattern and signalized intersection characteristics to predict accidents at signalized intersections in a city’s arterial network. An analysis of historical accident data at selected intersections within the City of Norfolk indicated that in addition to traffic volume, other controllable factors contributed to traffic accidents at specific intersections. The structural factor included variables such as area topography, lane patterns, type of road signs, turning lanes, etc., the administrative factor included variables such as signal types, signal polices, road closures, etc., and maintenance factor included variables such as road conditions, condition of the signals, condition of road signs, etc.

Statistical models are helpful for identifying factors associated with motor vehicle crashes, despite their vast limitations. To this end, many studies have developed crash models focused on predicting total crashes (totals, fatalities, injuries, etc.) for assessing the safety effects of various factors. While these models have great utility, they are limited because they fail to relate crash types with roadway, traffic, and environmental factors, as discussed in Kim et al. (2006).

D. Kim et al. (2007) concluded that many probability models of crashes have revealed that AADT is a significant and important predictor of crash frequencies; however, AADT variables were not found to have significant effects on the probability that certain types of crashes will occur, as opposed to expectation. In order to explore the safety effects of AADT on the probabilities of crash types, a variety of transformation on AADT variables (i.e. categorical variable, indicator variable, log of AADT, etc.) were conducted in different ways, but all expressions of AADT variables were found to be non-significant.
for predicting crash types. It is known that average daily traffic volumes are strongly associated with crash frequencies because greater volumes lead to greater opportunities for collisions—and so this result is generally troubling.

**DATA COLLECTION AND PREPARATION**

In Abu Dhabi, sever crash data are collected by the Traffic and Patrols Directorate. The data include details regarding the vehicle and driver characteristics, level of injury, environmental condition, lighting, road type, crash time, causes and type of crash, number of injuries and fatalities. A total of 843 crashes involving signalized intersections from year 2008 to 2013 were isolated. To isolate signalized intersections in the crash database, some fields were used including crashes’ description, location description, intersected roads, and intersections type.

**ANALYSIS AND RESULTS**

**Safety Evaluation**

Table (1) show that signalized intersections crashes in Abu Dhabi City increased by 130% between years 2008 and 2013. Figure (1) shows a serious increase in the number of signalized intersections crashes (This is due to the economic and urban development in Abu Dhabi city, which started in 2005, that resulting significant increase in population and thus high number of driving licenses owners and vehicles on the road network), where the percentage of those crashes increased between 2008 and 2010 by about 88%, and then declined in 2011 by 8% from 2010. However, that the number of such crashes quickly rebounded seriously and rate reached 33% between 2011 and 2013. The table show that the signalized intersections crashes were formed 8% from the total number of crashes in year 2008 that percentage was increased in 2013 up to 38%.

Table (1) also show that despite of decrease in Total number of crashes' fatalities & serious injuries in Abu Dhabi City during study period by 45%, the number of fatalities & serious injuries at signalized intersection's crashes increased by 29%. The same observation can be concluded for pedestrians’ fatalities at signalized intersection's crashes that are increased by 33%.

<table>
<thead>
<tr>
<th>Table (1): Signalized Intersections Crashes, Abu Dhabi City, 2008-2013</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Abu Dhabi City</strong></td>
</tr>
<tr>
<td>Total number(a) of severe crashes(b)</td>
</tr>
<tr>
<td>Severe crashes at signalized intersections</td>
</tr>
<tr>
<td>Percentage of Total</td>
</tr>
<tr>
<td>Total number of crashes' fatalities &amp; injuries</td>
</tr>
<tr>
<td>Number of fatalities &amp; injuries at signalized intersection's crashes</td>
</tr>
<tr>
<td>Percentage of Total</td>
</tr>
<tr>
<td>Total number of crashes' fatalities</td>
</tr>
<tr>
<td>Number of fatalities at signalized intersection's crashes</td>
</tr>
<tr>
<td>Percentage of Total</td>
</tr>
</tbody>
</table>
Total number of crashes' serious injuries<sup>(c)</sup> | 87 | 99 | 73 | 61 | 65 | 55  
---|---|---|---|---|---|---  
Number of serious injuries at signalized intersection's crashes | 9 | 9 | 23 | 9 | 15 | 13  
Percentage of Total | 10% | 9% | 32% | 15% | 23% | 24%  
Total number of crashes' fatalities & serious injuries | 128 | 134 | 103 | 87 | 99 | 71  
Number of fatalities & serious injuries at signalized intersection's crashes | 14 | 11 | 28 | 11 | 25 | 18  
Percentage of Total | 11% | 8% | 27% | 13% | 25% | 25%  
Total number of pedestrians fatalities | 20 | 25 | 15 | 16 | 10 | 10  
Number of pedestrians fatalities at signalized intersection's crashes | 3 | 1 | 1 | 2 | 2 | 4  
Percentage of Total | 15% | 4% | 7% | 13% | 20% | 40%  

(a) Total number is the whole number and statistics of severe crashes and its results been in Abu Dhabi City from 2008 to 2013.
(b) Severe Crashes are those crashes resulted in injuries or fatalities.
(c) Serious injuries defined by physician and often those determined to injury recovery period exceeding 21 days, it is cause failure of the injured from work, who should be under medical observation during treatment period.

Signalized intersections’ crashes in Abu Dhabi City varied during study period, which pedestrian crashes, deterioration, and collision (of all kinds). However, the most prominent types of crashes during the study period (2008 to 2013) were as follows showing in figure (2):
Right angle collision: formed 63% of the total crashes at signalized intersections. Those crashes increased significantly.

Side swipe collision: formed 14% of the total crashes at signalized intersections. Those crashes decline by a large margin during the study period.

Pedestrian crashes: Were 9% of the total crashes at signalized intersections. Those crashes are relatively increased.

![FIGURE 2: The most important types of signalized intersections crashes in Abu Dhabi City](image)

It is noted that the ratio of signalized intersections crashes in Abu Dhabi City were close during the week days, ranging between 12% and 14%, but they are increased through weekend where the percentage of accidents during those days about 16% of the total crashes. Also, It is noted that the ratio of signalized intersections crashes occurring during evening (around 38%), but the morning and afternoon periods also have a large number of crashes formed about 50% combined. Thus it can be concluded that the peak periods in Abu Dhabi has 88% of the total number of signalized intersections crashes.

Statistics showed that 32% of signalized intersections crashes caused by drivers whose aged between (31 – 40 years), while drivers who are aged between (25 - 30 years) caused 26% of those crashes. Moreover, 82% of those drivers were males, and 70% of those drivers have experience between 1 and 10 years.

Signalized intersections crashes in Abu Dhabi City varied, but crossing red light signal highlighted as the main reason by 77% of those crashes. Crashes occurred due to Driving under the influence of intoxicating or anesthetic formed 4.7% of those crashes, the Non-compliance with road marking formed 4.2%, while Not leaving enough distance formed 3.1% of signalized intersections crashes in Abu Dhabi City.
Figure (3) shows a serious increase in the number of signalized intersections crashes caused by crossing red light signal, where the percentage of those crashes increased between 2008 and 2010 by about 126%, and then declined in 2011 by 3% from 2010. However, that the number of such crashes quickly rebounded seriously and rate reached 26% between 2011 and 2013.

Figure 3: Signalized intersections crashes in Abu Dhabi City (2008-2013) caused by crossing red light signal

Until 2009 all the signals in Abu Dhabi city were working on (Split Traffic Signal Phasing) - Split phasing represents an assignment of the right-of-way to all movements of a particular approach, followed by all of the movements of the opposing approach), with the increase in the number of vehicles and traffic jams on the intersections (Lead-Lag Signal Phase) - The Lead-Lag left turn phase sequence has one of the opposing left turn phases starting and operating concurrent with its respective through movements, and the other left turn phase starting subsequent to the opposing through movement and ending simultaneously with its concurrent through movement) system applied at 38 intersections to give more time for forward direction and increase intersections capacity. Statistics showed that up to the year 2009 signalized intersections crashes in Abu Dhabi City at intersections managed by (Split Traffic Signal Phasing) system and others later managed by (Lead-Lag Signal Phase) system were close. But by the application of the new system (Lead-Lag Signal Phase) on some intersections crashes dramatically increased up to 104% between 2009 and 2010 on that intersections, then declined in 2011 by 19% from 2010. However, that the number of such crashes quickly rebounded seriously and the rate reached 38% between 2011 and 2013 as shown in figure (4).
The total number of signalized intersections crashes’ victims during study period (2008-2013) were 1635 victim, distributed as shown in Figure (5) below.

Figure 5: Number of fatalities & injuries at signalized intersection's crashes in Abu Dhabi City (2008-2013)

Right angle collision crashes caused 70% of the victims and left 16 fatalities (55% of total fatalities). Side swipe collision crashes caused 13% of the victims, as well as
pedestrian crashes caused 5% of the victims and left 13 fatalities (45% of total fatalities). Also signalized intersection crashes occurred at evening period caused 41% of the victims and left 17 fatalities (59% of total fatalities). Statistics showed that 33% of signalized intersections crashes victims were between (31 – 40 years), while 26% of them were between (25 - 30 years). Moreover, 80% of those victims were males. Since crossing red light signal has emerged as the main reason for 77% of signalized intersection crashes in Abu Dhabi City, has resulted in an 83% of victims and caused the death of 19 people.

Crashes occurred on signalized intersections managed by (Lead-Lag Phase) system caused 64% of the victims and left 16 fatalities (55% of total fatalities).

**Model**

One of the primary objectives of this study was to identify the important driver, vehicle, road, environment, and crash related factors that influence the severity of vehicle crashes. First, important crash characteristics related to vehicle crashes have been identified and then how these factors affect the severity of crashes were analyzed by developing binary logit models. Logistic regression analysis is used to investigate the association between a number of explanatory variables and crash severity at signalized intersections in Abu Dhabi City. The factors identified in this study are expected to help developing potential countermeasures which will ultimately reduce the severity as well as the number of crashes.

(Xuesong Wang, 2006) during his safety analyses at signalized intersections considering spatial, temporal, and site correlation mentioned that it is well accepted that traffic crashes at signalized intersections are complicated events which involve the interaction between the driver, vehicle, roadway, traffic, and the environment. However, the relationship between crash occurrence and these contributing factors is still not clear. Logistic regression or other relevant statistical methods are common in severity modeling. Several studies have adopted this kind of models to examine the association between crash characteristics and crash severity. Liu et al.2009 in a study examining different factors affecting crash severity on gravel roads used binary logit model. Young et al. 2007 developed binary logit model to estimate the relationship between wind speed and overturning truck crashes. In a study to determine the effectiveness of seat belts in reducing injuries, Ratnayake, 2006 used binary logit model. In another study done by Zhu et al., 2010, binary logit model was developed in predicting fatal crashes for two lane rural highways in the Southeastern United States. Different driver, vehicle, road, crash, and environment related factors that influence crash severity are identified by using binary logit models conducted by S. Dissanyake and U. Roy, 2013.

**Methodology**

**Variable Selection**

Firstly, it was tried to include as many variables (vehicle, driver, roadway, and environment) as possible for the modeling. At that stage we considered the fact that the quality of the modeling could be expected to increase to a certain level once the number of variables increases. Secondly, selection of the variables was carried out depending on
previous studies and the assumption that a particular variable would affect the severity of signalized intersections crashes. The descriptions of 15 explanatory variables that are considered for the modeling are provided along with their statistics in Table 2. All the explanatory variables are binary. Binary variables take the form of either 0 or 1; for example, if a crash occurs on CBD, the variable CBD has been assigned “1” as its value; otherwise “0” is assigned to this variable. Two binary logistic regression models were developed by considering crash severity as the response variable and the description of the models are as follows:

1) CRASH_SEVERITY1 (Binary response = 1 if the observation is a fatal crash, =0 otherwise i.e. serious, medium, and minor injury)
2) CRASH_SEVERITY2 (Binary response = 1 if the observation is a fatal or serious injury crash, =0 otherwise i.e. medium and minor injury)

<table>
<thead>
<tr>
<th>Variable</th>
<th>Mean</th>
<th>Std. Deviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>LEAD_LAG_PHASING</td>
<td>0.58</td>
<td>0.493</td>
<td>=1 if the signal phasing is Lead-Lag, =0 otherwise</td>
</tr>
<tr>
<td>NO_OF_APPROACHES</td>
<td>0.90</td>
<td>0.306</td>
<td>=1 if the number of approaches is 4, =0 otherwise</td>
</tr>
<tr>
<td>CBD</td>
<td>0.36</td>
<td>0.480</td>
<td>=1 if crash occurred on CBD signalized intersections, =0 otherwise</td>
</tr>
<tr>
<td>ROAD_SPEED</td>
<td>0.12</td>
<td>0.322</td>
<td>=1 if crash occurred on road speed ≥80 km/h, =0 otherwise</td>
</tr>
<tr>
<td>RED_LIGHT_CROSSING</td>
<td>0.78</td>
<td>0.416</td>
<td>=1 if the crash occurred due to red light crossing, =0 otherwise</td>
</tr>
<tr>
<td>CAUSED_VEHICLE</td>
<td>0.90</td>
<td>0.297</td>
<td>=1 if the vehicle caused crash at signalized intersection is light vehicle, =0 otherwise</td>
</tr>
<tr>
<td>GENDER_OF_CAUSES(a)</td>
<td>0.82</td>
<td>0.386</td>
<td>=1 if the driver is a male, =0 otherwise</td>
</tr>
<tr>
<td>CAUSES_EXPERIENCE</td>
<td>0.70</td>
<td>0.459</td>
<td>=1 if the driver caused crash at signalized intersection experience between (1-10)years, =0 otherwise</td>
</tr>
<tr>
<td>ALCOHOL</td>
<td>0.05</td>
<td>0.213</td>
<td>=1 if the driver is under influence, =0 otherwise</td>
</tr>
<tr>
<td>DAYLIGHT</td>
<td>0.56</td>
<td>0.497</td>
<td>=1 If crashes occur during daylight, =0 otherwise</td>
</tr>
<tr>
<td>DR_TOO_FAST</td>
<td>1.00</td>
<td>0.000</td>
<td>=1 If the driver is too fast for conditions or exceeded posted speed limit, =0 otherwise</td>
</tr>
<tr>
<td>LICST</td>
<td>0.76</td>
<td>0.426</td>
<td>=1 If the driver has the license in Abu Dhabi, =0 otherwise</td>
</tr>
<tr>
<td>OLDDR</td>
<td>0.02</td>
<td>0.123</td>
<td>=1 if the driver is 60 years or older, =0 otherwise</td>
</tr>
<tr>
<td>YOUNGDR</td>
<td>0.19</td>
<td>0.389</td>
<td>=1 if the driver is between 16 and 24 years, =0 otherwise</td>
</tr>
<tr>
<td>WEEKEND</td>
<td>0.30</td>
<td>0.457</td>
<td>=1 If the crashes occur during weekend (Friday &amp; Saturday), =0 otherwise</td>
</tr>
</tbody>
</table>

(a) GENDER_OF_CAUSES means gender of those who caused the traffic crash.
Logistic Regression
As one of the aims of the study was to develop models to predict the severity of signalized intersections crashes, logistic regression was identified as the most suitable approach to identify the important factors. Therefore, binary logit model has been identified as the most suitable approach in this study. In case of binary logistic regression model, the response variable, $y$ takes the form of either of the two binary values (0 or 1). For $k$ explanatory variables and $i=1, 2, 3, \ldots, n$ individuals, the model takes the form as follows

$$
\log \left[ \frac{P_i}{1 - P_i} \right] = \alpha + \beta_1 x_{i1} + \beta_2 x_{i2} + \cdots + \beta_k x_{ik}
$$

$P_i = \text{Prob. ( } y_i = y_1 \mid X_i )$ is the response probability to be modeled, and $y_1$ is the first ordered level of $y$,

$\alpha = \text{Intercept parameter},$

$\beta = \text{Vector of slope parameters},$

$X_i = \text{Vector of explanatory variables}.$

The odds ratio for dichotomous explanatory variable, $x$, which takes value 1 or 0 (with 1 meaning that the event will certainly occur and 0 meaning that the event will definitely not occur) can be represented as the ratio of the expected number of times that an event will occur ($x = 1$) to the expected number of times it will not occur ($x = 0$). This can be illustrated by the formula below

$$
\text{OR} = \frac{\pi (1)/[1 - \pi (1)]}{\pi (0)/[1 - \pi (0)]}
$$

Where,

$\text{OR} = \text{Odd Ratio}$

$\pi (1)/[1 - \pi (1)] = \text{Probability that the event will occur when } x=1$

$\pi (0)/[1 - \pi (0)] = \text{Probability that the event will occur when } x=0$

Model Analysis results
The results of the two crash severity models that are developed are presented in Tables (3.a) and (4.a). The model uses 843 crash records in total. The first model, where the response variable is CRASH_SEVERITY1 (fatalities), has four explanatory variables as significant. The coefficients of explanatory variables are directly related to the probability of having a more severe crash. The variable with positive coefficient denotes the increasing probability of a certain crash severity and vice versa. All the four significant independent variables are found to have positive coefficients, which mean that the probability of a fatal crash is likely to increase when one or more of these 4 factors are involved.
Explanatory variables are Driver related (DR_TOO_FAST, LICST, ALCOHOL), and road related (ROAD_SPEED). The odds ratios presented measure the amount by which the crash severity increases. Taking an example of the explanatory variable ROAD_SPEED, which has an odds ratio of 3.127 for the first model, it can be stated that the probability of fatal crash tends to be 3.127 times higher when road speed at intersection approach is equal or greater than 80 km/hr, assuming that rest of the factors remains the same.

| Table (3.a): Estimation of the CRASH_SEVERITY1 model for single vehicle crashes at signalized intersections in Abu Dhabi City. |
|---------------------------------|------------|--------|--------|--------|----------------|
| B                              | S.E.       | Wald   | df    | Sig.   | Odds Ratios  |
| DAYLIGHT                       | -0.709     | 0.466  | 2.316 | 0.128  | 0.492        |
| DR_TOO_FAST                    | 3.425      | 1.184  | 8.369 | 0.004  | 30.721       |
| ROAD_SPEED_A                   | 1.140      | 0.615  | 3.432 | 0.064  | 3.127        |
| LICST                          | -0.820     | 0.465  | 3.113 | 0.078  | 0.440        |
| LEAD_LAG_PHASING               | -0.219     | 0.505  | 0.189 | 0.664  | 0.803        |
| NO_OF_APPROACHES               | 0.362      | 0.873  | 0.172 | 0.678  | 1.436        |
| CBD                            | 0.481      | 0.521  | 0.854 | 0.355  | 1.618        |
| RED_LIGHT_CROSSING             | 1.164      | 1.072  | 1.179 | 0.277  | 3.204        |
| CAUSED_VEHICLE                 | -0.025     | 0.684  | 0.001 | 0.970  | 0.975        |
| GENDER_OF_CAUSES               | -0.044     | 0.592  | 0.005 | 0.941  | 0.957        |
| CAUSES_EXPERIENCE              | -0.244     | 0.479  | 0.260 | 0.610  | 0.784        |
| ALCOHOL                        | 2.258      | 1.208  | 3.496 | 0.062  | 9.562        |
| OLDDR                          | -17.191    | 10759.233 | 0.000 | 0.999  | 0.000        |
| WEEKEND                        | 0.273      | 0.450  | 0.367 | 0.545  | 1.313        |
| YOUNGDR                        | -0.161     | 0.591  | 0.074 | 0.786  | 0.852        |
| Constant                       | -4.428     | 1.631  | 7.374 | 0.007  | 0.012        |

Depending on the first model, another model been fitted for the same response variable found that the probability of a fatal crash at signalized intersections is likely to increase when the driver has license issued from Abu Dhabi, and he was driving too fast or exceeded posted speed limit on intersection located in road which its speed ≥80 km/h, and crash occurred during daylight. (Table 3.b)

| Table (3.b): Estimation of the CRASH_SEVERITY1 (2nd model) for single vehicle crashes at signalized intersections in Abu Dhabi City. |
|---------------------------------|------------|--------|--------|--------|----------------|
| B                              | S.E.       | Wald   | df    | Sig.   | Odds Ratios  |
| DAYLIGHT                       | -0.847     | 0.443  | 3.650 | 0.056  | 0.429        |
| DR_TOO_FAST                    | 2.295      | 0.618  | 13.789 | 0.000  | 9.924        |
| ROAD_SPEED_A                   | 0.896      | 0.534  | 2.812 | 0.094  | 2.450        |
| LICST                          | -0.813     | 0.449  | 3.285 | 0.070  | 0.444        |
| Constant                       | -2.949     | 0.412  | 51.190 | 0.000  | 0.052        |
The second model, where the response variable is CRASH_SEVERITY2 (crash severity is fatal or serious injuries), has three explanatory variables as significant. Explanatory variables are driver related (GENDER_OF_CAUSES, DR_TOO_FAST), and environment related (DAYLIGHT). All the three significant independent variables are found to have positive coefficients, which mean that the probability of a fatal or serious crash is likely to increase when one or more of these 3 factors are involved.

Table (4.a): Estimation of the CRASH_SEVERITY2 model for single vehicle crashes at signalized intersections in Abu Dhabi City.

<table>
<thead>
<tr>
<th></th>
<th>B</th>
<th>S.E.</th>
<th>Wald</th>
<th>df</th>
<th>Sig.</th>
<th>Odds Ratios</th>
</tr>
</thead>
<tbody>
<tr>
<td>DAYLIGHT</td>
<td>-0.475</td>
<td>0.241</td>
<td>3.904</td>
<td>1</td>
<td>0.048</td>
<td>0.622</td>
</tr>
<tr>
<td>DR_TOO_FAST</td>
<td>1.817</td>
<td>0.547</td>
<td>11.041</td>
<td>1</td>
<td>0.001</td>
<td>6.152</td>
</tr>
<tr>
<td>ROAD_SPEED_A</td>
<td>0.436</td>
<td>0.356</td>
<td>1.494</td>
<td>1</td>
<td>0.222</td>
<td>1.546</td>
</tr>
<tr>
<td>LICST</td>
<td>-0.219</td>
<td>0.266</td>
<td>0.680</td>
<td>1</td>
<td>0.410</td>
<td>0.803</td>
</tr>
<tr>
<td>LEAD_LAG_PHASING</td>
<td>0.020</td>
<td>0.261</td>
<td>0.006</td>
<td>1</td>
<td>0.938</td>
<td>1.021</td>
</tr>
<tr>
<td>NO_OF_APPROACHES</td>
<td>0.426</td>
<td>0.472</td>
<td>0.815</td>
<td>1</td>
<td>0.367</td>
<td>1.532</td>
</tr>
<tr>
<td>CBD</td>
<td>0.059</td>
<td>0.273</td>
<td>0.046</td>
<td>1</td>
<td>0.830</td>
<td>1.060</td>
</tr>
<tr>
<td>RED_LIGHT CROSSING</td>
<td>0.180</td>
<td>0.379</td>
<td>0.226</td>
<td>1</td>
<td>0.635</td>
<td>1.198</td>
</tr>
<tr>
<td>CAUSED_VEHICLE</td>
<td>-0.382</td>
<td>0.351</td>
<td>1.184</td>
<td>1</td>
<td>0.276</td>
<td>0.683</td>
</tr>
<tr>
<td>GENDER_OF_CAUSES</td>
<td>0.685</td>
<td>0.375</td>
<td>3.334</td>
<td>1</td>
<td>0.068</td>
<td>1.983</td>
</tr>
<tr>
<td>CAUSES_EXPERIENCE</td>
<td>-0.324</td>
<td>0.258</td>
<td>1.582</td>
<td>1</td>
<td>0.209</td>
<td>0.723</td>
</tr>
<tr>
<td>ALCOHOL</td>
<td>0.660</td>
<td>0.558</td>
<td>1.396</td>
<td>1</td>
<td>0.237</td>
<td>1.934</td>
</tr>
<tr>
<td>OLDDR</td>
<td>0.668</td>
<td>0.817</td>
<td>0.667</td>
<td>1</td>
<td>0.414</td>
<td>1.950</td>
</tr>
<tr>
<td>WEEKEND</td>
<td>0.290</td>
<td>0.244</td>
<td>1.418</td>
<td>1</td>
<td>0.234</td>
<td>1.337</td>
</tr>
<tr>
<td>YOUNGDR</td>
<td>0.164</td>
<td>0.295</td>
<td>0.309</td>
<td>1</td>
<td>0.579</td>
<td>1.178</td>
</tr>
<tr>
<td>Constant</td>
<td>-2.663</td>
<td>0.796</td>
<td>11.184</td>
<td>1</td>
<td>0.001</td>
<td>0.070</td>
</tr>
</tbody>
</table>

Also depending on the second model, another model been fitted for the same response variable found that the probability of a fatal or severe crash at signalized intersections is likely to increase when the driver been male, and he was driving too fast or exceeded posted speed limit, and crash occurred during daylight. (Table 4.b)

Table (4.b): Estimation of the CRASH_SEVERITY2 (2nd model) for single vehicle crashes at signalized intersections in Abu Dhabi City.

<table>
<thead>
<tr>
<th></th>
<th>B</th>
<th>S.E.</th>
<th>Wald</th>
<th>df</th>
<th>Sig.</th>
<th>Odds Ratios</th>
</tr>
</thead>
<tbody>
<tr>
<td>DAYLIGHT</td>
<td>-0.495</td>
<td>0.229</td>
<td>4.660</td>
<td>1</td>
<td>0.031</td>
<td>0.610</td>
</tr>
<tr>
<td>DR_TOO_FAST</td>
<td>1.664</td>
<td>0.415</td>
<td>16.063</td>
<td>1</td>
<td>0.000</td>
<td>5.283</td>
</tr>
<tr>
<td>GENDER_OF_CAUSES</td>
<td>0.744</td>
<td>0.368</td>
<td>4.095</td>
<td>1</td>
<td>0.043</td>
<td>2.105</td>
</tr>
<tr>
<td>Constant</td>
<td>-2.618</td>
<td>0.365</td>
<td>51.332</td>
<td>1</td>
<td>0.000</td>
<td>0.073</td>
</tr>
</tbody>
</table>

We tried to fit a model that discovers the effect of lead lag phasing and any other explanatory variable on both type of response variables (CRASH_SEVERITY1 and CRASH_SEVERITY2). The results showed that there are no significant relationship between lead lag phasing and both type of response variables. So, more 3 explanatory variables (related to crash type) that are considered for the modeling are provided. All the explanatory variables are binary.
SIDESWIPE_COLLISIONS, and RIGHTANGLE_COLLISIONS combined with LEAD_LAG_PHASING and other explanatory variables are used to fit a model and find if there are significant relationships with both type of response variables (CRASH_SEVERITY1 and CRASH_SEVERITY2).

The first model, where the response variable is CRASH_SEVERITY1 (crash severity is fatal), has three explanatory variables as significant. Explanatory variables as shown in table (5) are signal system related LEAD_LAG_PHASING, driver related ALCOHOL and crash type related PEDESTRIAN_CRASHES, which have an odds ratio of 2.566, 3.137, and 20.214. it can be observed from this model that the probability of fatality at signalized intersections crashes in Abu Dhabi City increased when the signal managed by lead lag phasing system and driver is under influence while crash type been pedestrian crash. Also it can be stated that the probability of fatal crash tends to be 2.566 times higher when signal managed by lead lag phasing system, at the same time probability of fatal crash tends to be 3.137 times higher when driver been under influence, and 20.130 times higher when crash resulted pedestrian victims.

The second model, where the response variable is CRASH_SEVERITY2 (crash severity is fatal or serious injuries), has no explanatory variables as significant.

Table (5): Estimation of the CRASH_SEVERITY1 model that discovers the effect of lead lag phasing and any other explanatory variable in Abu Dhabi City.

<table>
<thead>
<tr>
<th></th>
<th>B</th>
<th>S.E.</th>
<th>Wald</th>
<th>df</th>
<th>Sig.</th>
<th>Odds Ratios</th>
</tr>
</thead>
<tbody>
<tr>
<td>LEAD_LAG_PHASING</td>
<td>0.943</td>
<td>0.493</td>
<td>3.650</td>
<td>1</td>
<td>0.056</td>
<td>2.566</td>
</tr>
<tr>
<td>ALCOHOL</td>
<td>1.143</td>
<td>0.701</td>
<td>2.660</td>
<td>1</td>
<td>0.103</td>
<td>3.137</td>
</tr>
<tr>
<td>PEDESTRIAN_CRASHES</td>
<td>3.006</td>
<td>0.485</td>
<td>38.391</td>
<td>1</td>
<td>0.000</td>
<td>20.214</td>
</tr>
<tr>
<td>Constant</td>
<td>-4.986</td>
<td>0.508</td>
<td>96.318</td>
<td>1</td>
<td>0.000</td>
<td>0.007</td>
</tr>
</tbody>
</table>

Conclusion

The study investigated the factors that affect the traffic crashes and severity at Abu Dhabi signalized intersection. Signalized intersections crashes in Abu Dhabi City increased by 130% between years 2008 and 2013. The number of fatalities & serious injuries at signalized intersection's crashes increased by 29% during study period, the same observation can be concluded for pedestrians’ fatalities at signalized intersection's crashes that are increased by 33%.

It is noted that the ratio of signalized intersections crashes in Abu Dhabi City were close during the week days, but they are increased through weekend. Also, it can be concluded that the peak periods in Abu Dhabi has 88% of the total number of signalized intersections crashes. More than one third of signalized intersections crashes caused by drivers whose aged between (31 – 40 years), moreover, 82% of those drivers were males, and two – third of them have experience between 1 and 10 years. Signalized intersections crashes in Abu Dhabi City varied, but crossing red light signal highlighted as the main reason by 77% of those crashes. The implementation of (Lead-Lag Signal Phase) system on some intersections increased crashes dramatically.

The total number of signalized intersections crashes’ victims during study period (2008-2013) was 1635 victims. Right angle collision crashes caused 70% of the victims and resulted 16 fatalities, as well as pedestrian crashes caused 5% of the victims and left 13
fatalities. Also signalized intersection crashes occurred at evening period caused 41% of the victims and left 17 fatalities. The analysis showed that one third of signalized intersections crashes victims were between (31 – 40 years). Moreover, 80% of those victims were males. Since crossing red light signal has emerged as the main reason for more than three quarters of signalized intersection crashes in Abu Dhabi City, it has resulted in an 83% of victims and caused the death of 19 people. Crashes occurred on signalized intersections managed by (Lead-Lag Phase) system caused 64% of the victims and left 16 fatalities (55% of total fatalities).

The study developed binary logit model in order to determine the important factors associated with the severity of signalized intersections crashes. Two models had been established by using 843 crash records, and 8 explanatory variables were used in this study to identify how they influenced signalized intersections crash severity. 6 variables appeared to be positively associated with signalized crash severity for the two models; this means that they increase the severity of signalized intersection crashes. These variables are driver related factors (which were driving too fast, licensing Emirate, alcohol involvement, driver gender), road related variable which was speed, and environment related variable which was daylight.

We tried to fit a model that discovers the effect of lead lag phasing and any other explanatory variable on crash severity. The results showed that the probability of fatality at signalized intersections crashes in Abu Dhabi City increased when the signal managed by lead lag phasing system and driver is under influence while crash type been pedestrian crash.

Depending on the above results we recommend for application plan to reduce fatalities and serious injuries resulting from signalized intersections’ crashes within Abu Dhabi City by implementation of an integrated system that takes into account: firstly, Improve engineering and construction determinants for signalized intersections that adversely affect their traffic safety performances. Secondly, intensify the use of automated and In-presence enforcement at signalized intersections. Along with intensify awareness programs directed to the target groups, and continuous evaluation of performance and results, while mitigate whenever the need arises.

At the same time there is need for evaluate the use of (Lead-Lag Signal Phase) instead of (Split Phasing) and its impact on intersections’ traffic safety performance that use them. Intersection surveillance system has been implemented in the emirate of Abu Dhabi since 2012 and is considered a comprehensive project which aims to enhance road safety. Under this five-year scheme cameras are being installed at 150 intersections in Abu Dhabi, Al Ain and the Western Region. Traffic cameras use infrared technology without flashing to nab motorists who run red lights, exceed speed limits, misuse traffic lanes, stop vehicles on pedestrian crossing lines, and catch speeders who illegally overtake other vehicles at intersections in addition to capturing vehicles that turn illegally or make a U-turn to the left from the wrong lane. The cameras are fully equipped to cover five lanes in all directions and are able to read vehicles plates in addition to counting the number of cars on the road at any given time. The instruments are also capable of assessing the average speed on the road and the number of pedestrians and the direction in which they are moving.
REFERENCES


The use of ceramic waste materials as fine aggregates in Hot Mix Asphalt (HMA)

1Dhieyatul Husna Ismail,
2Ratnasamy Muniandy,
3Salihudin Hassim

Graduate Student1, University Putra Malaysia (UPM), MALAYSIA
Email: dhieyatul@gmail.com
Professor2, University Putra Malaysia (UPM), MALAYSIA
Email: ratnas@upm.edu.my
Associate Professor3, University Putra Malaysia (UPM), MALAYSIA
Email: hsalih@upm.edu.my

ABSTRACT

The use of crushed ceramic waste in road construction is an attractive option in order to increase usage of industrial waste and enhance the properties of asphalt mixture. Laboratory tests were conducted to evaluate the feasibility of incorporating the waste of ceramic tile from tiling manufacturing into pavement material. In this study, crushed ceramic tiles were incorporated in fine aggregate of asphalt mixtures. Only size of 5.0 mm down to filler of crushed ceramic tiles were collected and added to asphalt mixture to replace fine aggregates. The replacement was done proportionally by 0, 20, 40, 60, 80 and 100 percent by weight of aggregates. The results of this study indicated that the use of ceramic tile can be potentially used in HMA mixture. Physical properties test for ceramic tile wastes were fulfilled the requirement. The Marshall stability showed an increment about 0.25 and resilient modulus test was improved up to 13.5% compared to control sample. All proportion of samples was ranked and 20% of ceramic was recommended to be used in Hot Mix Asphalt (HMA) mixture.

Keywords: Ceramic waste aggregate, fine aggregate replacement, Marshall stability, resilient modulus, hot mix asphalt

1. INTRODUCTION

Ceramic tile factory in Malaysia is facing serious problems of dumping and management of wastes ceramic tile. The rejected tiles of various types and sizes go to waste with no intention to recycle in any form. Approximately 10% of total production per year has gone to waste. The crushing process which cost million ringgit per ton is a real challenge to the industry to dispose this waste properly. After all, dumping constitute significant visual impact and environmental degradation. The use of ceramic tile waste in construction not only reduces the amount of dumping waste but can earn monetary saving from the disposal process. The costly crushing process is a real challenge to the ceramic tile industry to dispose the waste properly but through the idea of utilization ceramic tile in construction, it directly can solve their problem.

A review of earlier research showed that ceramic waste has been used in construction specifically in alternate of aggregate and filler. A study has been done by Huang et al. (2009) on the effect of ceramic waste material from automotive manufacturing used as filler incorporated in Portland cement and Asphaltic concrete. The ground ceramic waste has been substituted the natural sand in concrete specimen and also incorporated as mastic in asphalt binder. Slump test and compressive test have been done for concrete specimens and dynamic shear rheometer (DSR) test has been done for asphalt binders and Dynamic modulus, flow number and indirect tensile test have been done for HMA mixtures. The results showed an improvement to slump test as well as compressive test for concrete. It also showed an improvement in rutting resistance and permanent deformation for asphalt and HMA mixture.

Ratnasamy et. al. (2009) also carried out many studies on the use of ceramic tile waste as a filler for asphalt mixture. The study started with the initial investigation on feasibility of utilizing ceramic waste in asphalt binder which focuses into the physical and chemical analysis as well the composition of the material. The results had been check with the standard specification. Then, the study was continued to explore the effect of ceramic tile waste as a filler to asphalt binder in permanent deformation, indirect tensile test, moisture induced damage, dynamic modulus test and dynamic shear rheometer (DSR) test. All the tests carried out showed an improvement in rutting, fatigue and permanent deformation. However, ceramic glaze usually exists
on the surface of ceramic waste materials especially on ceramic tile and sanitary ware industry (Medina, 2012). This glaze prevents interfacial adhesion and bonding between asphalt and ceramic waste aggregates in the mixture.

The study on ceramic waste incorporated in concrete and asphalt mixture were not stopped to that. Studies on ceramic material are a kind of inorganic, nonmetallic materials produced by the action of heat and subsequent cooling. Ceramic also had a thermal resistance and insulator properties which can stand higher temperature with low thermal conductivity. Due to that, a study has been done by Feng (2013) to evaluate the thermal performance when incorporated ceramic waste in wearing layer of asphalt pavement. It showed that substitution of ceramic waste aggregate can reduce thermal conductivities of asphalt mixtures and reducing the temperature gradient of pavement.

The study on ceramic waste incorporated in concrete and asphalt mixture were not stopped to that. Studies on ceramic material are a kind of inorganic, nonmetallic materials produced by the action of heat and subsequent cooling. Ceramic also had a thermal resistance and insulator properties which can stand higher temperature with low thermal conductivity. Due to that, a study has been done by Feng (2013) to evaluate the thermal performance when incorporated ceramic waste in wearing layer of asphalt pavement. It showed that substitution of ceramic waste aggregate can reduce thermal conductivities of asphalt mixtures and reducing the temperature gradient of pavement.

There were many studies have been done by replacing or substituting sand with ceramic waste in concrete and the use of ceramic waste as a filler or modifier for asphalt binder in pavement mixes. In this study, an attempt has been made to find out whether ceramic tile is suitable substitute for conventional crushed fine aggregate. The main objective is to evaluate the performance of asphalt mixture when ceramic waste aggregates were substituted. Only size of 5.0 mm down to filler of crushed ceramic waste were collected and added to asphalt mixture to replace fine aggregates. The replacement was done proportionally by 0, 20, 40, 60, 80 and 100 percent by weight of aggregates. This study has been carried out mainly to investigate the stability and flow value, the resilient modulus and void properties of the mixture.

2. MATERIALS AND METHODS

2.1 Materials and Sample Preparation

Ceramic wastes were obtained from ceramic tile manufacturing company, Kawasan Perindustrian Selayang, Batu Caves, Selangor. Waste ceramics collected from tile manufacturing factory. According to a survey carried out in 2012, a tile factory in Malaysia had produced 5% ceramic wastes and dumped without recycling to any form. In building construction, ceramic waste is produced on the transportation to the building site, on the execution of several construction elements (facades and partition walls, roofs and precast joist slabs) and on subsequent works, such as opening of grooves. The waste is regionally deposited in dumping grounds, without any separation or reuse. For this study, ceramic tiles were crushed by using portable crushing machine available in Universiti Putra Malaysia. The process of crushing is showed in Figure 1.

Figure 1. The process of raw ceramic waste crushing into small sizes
Dry and clean granite also used in this study for coarse aggregates. Granite maximum nominal size is 14 mm. The physical properties for ceramic waste and granite aggregate are given in Table 1. Although it was observed that the moisture absorption by the ceramic aggregate is slightly higher than granite, it is still lower than the maximum allowable of 2%. However further studies are expected to be carried out to investigate this issue. An asphalt binder of 80/100 penetration grade was used for mixture preparation. The basic properties for asphalt binder are shown in Table 2.

### Table 1. Physical Properties of Aggregates

<table>
<thead>
<tr>
<th>Properties</th>
<th>Standard</th>
<th>Type of Aggregate</th>
<th>Granite</th>
<th>Ceramic Waste</th>
</tr>
</thead>
<tbody>
<tr>
<td>Los Angeles Abrasion</td>
<td>ASTM C 131</td>
<td></td>
<td>20.12%</td>
<td>20.00%</td>
</tr>
<tr>
<td>Aggregate Impact Value</td>
<td>BS 812: Part 3</td>
<td></td>
<td>8.80%</td>
<td>4.30%</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>ASTM C127</td>
<td></td>
<td>2.612</td>
<td>2.381</td>
</tr>
<tr>
<td>Water Absorption</td>
<td>AASHTO T85</td>
<td></td>
<td>0.5%</td>
<td>1.038%</td>
</tr>
<tr>
<td>Flakiness Index</td>
<td>ASTM D 4791, BS 812</td>
<td></td>
<td>6.12%</td>
<td>95.56%</td>
</tr>
<tr>
<td>Elongation Index</td>
<td>ASTM D 4791, BS 812</td>
<td></td>
<td>0.07%</td>
<td>0%</td>
</tr>
<tr>
<td>Soundness Test</td>
<td>ASTM C88</td>
<td></td>
<td>0.12%</td>
<td>0.08%</td>
</tr>
</tbody>
</table>

### Table 2. Physical properties of Asphalt

<table>
<thead>
<tr>
<th>Type of Test</th>
<th>Standard used</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity at 25°C, (g/cm³)</td>
<td>ASTM D70</td>
<td>1.04</td>
</tr>
<tr>
<td>Penetration at 25°C, (0.1 mm), 100 g, 5s</td>
<td>ASTM D5</td>
<td>82</td>
</tr>
<tr>
<td>Softening point (R&amp;B), °C</td>
<td>ASTM D36</td>
<td>46.5</td>
</tr>
<tr>
<td>Viscosity at 135°C, Pascal second (Pa.s)</td>
<td>ASTM D4402</td>
<td>0.353</td>
</tr>
<tr>
<td>Viscosity at 165°C, Pascal second (Pa.s)</td>
<td>ASTM D4402</td>
<td>0.115</td>
</tr>
</tbody>
</table>

Note: 1 Pascal second (Pa.s) = 1000 centipoise (cP)

A crushed coarse and fine aggregate with maximum size of 14 mm was selected for dense graded asphalt mixture. The gradation and the corresponding mix designations are given in Table 3 and the resultant particle size distribution for HMA Mixture together with Public Works Department (PWD) specification limits are shown in Figure 2. Sieve analysis was carried out on a representative granite and ceramic waste. The dry sieve analysis was carried out according to ASTM D546 and AASHTO T37.

### Table 3. JKR Specification for Gradation ACWC 14 (HMA 14)

<table>
<thead>
<tr>
<th>Sieve size (mm)</th>
<th>Percentage passing (%)</th>
<th>Selected gradation</th>
<th>Recommended gradation limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>100</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>95</td>
<td>90-100</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>81</td>
<td>76-86</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>56</td>
<td>50-62</td>
<td></td>
</tr>
<tr>
<td>3.35</td>
<td>47</td>
<td>40-54</td>
<td></td>
</tr>
<tr>
<td>1.18</td>
<td>26</td>
<td>18-34</td>
<td></td>
</tr>
<tr>
<td>0.425</td>
<td>18</td>
<td>12-24</td>
<td></td>
</tr>
<tr>
<td>0.15</td>
<td>10</td>
<td>6-14</td>
<td></td>
</tr>
</tbody>
</table>
Total of six mix designs were made with the same blend of coarse aggregates but varied in the weight of fine aggregates. Table 4 shows the replacement of aggregate by percentage of weight. Samples made with granite became the control sample which was named as GC, and the samples named by 20 C to 100 C indicate the percentage of ceramic waste aggregate substituted in the sample. Control sample has 100% granite aggregate and substitution was started from 20%, 40%, 60%, 80% and 100% of ceramic tile waste. As mentioned above, the substitution of ceramic waste aggregate only involved fine aggregate which 5.00 mm to filler. The percentage indicated was from the total weight of aggregate gradation.

<table>
<thead>
<tr>
<th>Sieve size (mm)</th>
<th>GC</th>
<th>20 C</th>
<th>40 C</th>
<th>60 C</th>
<th>80 C</th>
<th>100 C</th>
</tr>
</thead>
<tbody>
<tr>
<td>14.00</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
</tr>
<tr>
<td>10.00</td>
<td>Granite</td>
<td>Granite</td>
<td>Granite</td>
<td>Granite</td>
<td>Granite</td>
<td>Granite</td>
</tr>
<tr>
<td>5.00</td>
<td>100%</td>
<td>20%</td>
<td>40%</td>
<td>60%</td>
<td>40%</td>
<td>100%</td>
</tr>
<tr>
<td>3.35</td>
<td>Granite</td>
<td>Ceramic waste</td>
<td>Ceramic waste</td>
<td>Ceramic waste</td>
<td>Ceramic waste</td>
<td>Ceramic waste</td>
</tr>
<tr>
<td>1.18</td>
<td>0.425</td>
<td>0.150</td>
<td>0.075</td>
<td>Pan</td>
<td>80%</td>
<td>60%</td>
</tr>
<tr>
<td>0.425</td>
<td>Ceramic waste</td>
<td>Ceramic waste</td>
<td>Ceramic waste</td>
<td>Ceramic waste</td>
<td>Ceramic waste</td>
<td>Ceramic waste</td>
</tr>
<tr>
<td>0.150</td>
<td>Granit</td>
<td>80%</td>
<td>60%</td>
<td>40%</td>
<td>20%</td>
<td></td>
</tr>
<tr>
<td>0.075</td>
<td>Granite</td>
<td>80%</td>
<td>60%</td>
<td>40%</td>
<td>20%</td>
<td></td>
</tr>
</tbody>
</table>

Note: % of aggregate weight

The mixtures were designed in accordance to the design procedures outlined by ASTM D 1559. The HMA mixtures were prepared at established mixing and compacting temperatures using Marshall Mix Design procedure to sustain heavy traffic using 50 blows per side. In this study triplicate specimens were prepared to evaluate the performance of substitution and replacement of ceramic on the stiffness and stability of HMA mixtures. The Marshall samples were tested by employing the following standard procedures: Bulk specific density (ASTM D2726), stability and flow test (ASTM D6927) and maximum theoretical specific gravity (ASTM D2041). The theoretical maximum density was determined through the rice method test. The samples were prepared according to the optimum asphalt content which were determined to be 5.26%, 5.26%, 5.33%, 5.4%, 5.8% and 5.81% for GC, 20C, 40C, 60C, 80C and 100C respectively.

2.3 Marshall Stability and Flow Test
The specimens were immersed in water at 60 °C for 30 min and then loaded to failure by using curved steel loading plates along with the diameter at a constant rate of compression of 51 mm/min. The stability and flow and the ratio of stability (kN) to flow (mm), stated as the Marshall quotient (MQ) and as indication of the stiffness of mixes were determined. It is well recognized that the MQ is a measure of the materials’ resistance to shear stresses, permanent deformation and rutting (10). High MQ values indicate a mix with high stiffness and with a greater ability to spread the applied load and resistance to creep deformation.

2.4 Resilient Modulus Test

An important input for the computation of flexible pavement responses under traffic loading is the resilient modulus of HMA (5). The Indirect Tensile Stiffness Modulus (ITSM) test is defined by BS DD 213 is nondestructive test. It has been identified as a potential means of measuring the stiffness modulus and effects of temperatures and load rate. Under uniaxial loading the stiffness modulus is generally defined as the ratio between the maximum stress and maximum strain (ASTM D 4123 and BS DD 213). The ITSM (Sm) in Mega Pascal (MPa) is calculated by the following equation:

\[
Sm = \frac{L(v + 0.27)}{D.t}
\]  

(1)

Where \( L \) is the peak value of the applied vertical load (N), \( D \) is the mean amplitude of the horizontal deformation obtained from 5 applications of the load pulse (mm), \( t \) is the mean thickness of the test specimen (mm), and \( v \) is the Poisson’s ratio (a value of 0.35 is normally used). The magnitude of the applied force conditioning pulses such that the specified target transient diametral deformation was achieved. The test was performed at 25 °C.

3. RESULTS AND DISCUSSIONS

3.1 Marshall stability and Flow test

The Marshal Stability results are presented as Figure 3. The specimens showed a dramatic trend in the stability results. It indicates substitution of 20% ceramic waste aggregates in the mixture were extremely increased the value of stability 0.25 higher than the control sample (GC). The value maintained higher for the samples of 40 C, 60 C and 80 C compared to control sample even though the value decreasing by increasing ceramic waste. It is considering that type of aggregate used in the mixture gave an effect to the Marshall Stability. Nevertheless, stability value of fully ceramic waste aggregates mixture (100C) shows lower value than the control sample. It is due to the smooth glazing of ceramic surface which prevent the bonding and adhesion between ceramic waste and asphalt. From the result, it proved that the combination of conventional aggregate and ceramic waste aggregate contributed good bonding that can resist higher loading to failure. Overall, the result of Marshall Stability for all proportions meets the requirements for the heavy traffic volume conditions.

![Figure 3. Stability value for all types of samples](image-url)
The stability test was done together with the flow measurement. The ratio of stability value to flow was given in Figure 4. MQ value shows greater increment up to 100% at 20 C samples compared to control sample. The MQ value slightly decreased at sample 40 C before maintains the same value at 60 C, 80 C and 100 C samples. Generally, the graph for MQ value indicates the same pattern with the stability graph where the value starts increasing when ceramic waste was utilized. The increasing of Marshall stability and Marshall Quotient (MQ) value indicates improved potential for carrying heavy load and resistance to rutting.

![Figure 4. MQ value for all types of samples](image)

### 3.2 Resilient Modulus Test

The Resilient Modulus results in mega Pascal (MPa) for all samples at 25 °C is showed in Figure 5.

![Figure 5. Resilient Modulus Test Results](image)

Generally the trend of resilient modulus results is quite similar with stability result. The resilient modulus for 20C has higher value 13.5% than control sample. After that, this value was decreasing by increasing of ceramic content. It has been expected since the performance of stability also dropped after 40 percent and more ceramic waste utilized to the mixture. This performance is due to the bonding and interlocking of aggregates that affect the workability of the mixture. As mentioned above, at certain amount of ceramic waste were developed a good bonding between ceramic-granite and can improved the elastic properties of the mixture. However, more ceramic waste was weaken the bonding and reduced the ability to retain higher load cycle frequency.
4. CONCLUSION

The objective of this study is to evaluate the suitability of using and incorporated ceramic tile waste into asphalt mixtures. From the physical properties test for ceramic waste, it found that all the result has fulfilled the minimum requirement of PWD Standard. This study also evaluated the performance of ‘modified’ asphalt mixture in carrying the loading and ability to resist the deformation.

The use of ceramic waste aggregate in replacement of part or fully granite has shown some improvements and acceptable results. This study can be concluded as listed as below:

- Both stability and resilient modulus results specifies that the best performing sample when comparing to granite control sample is 20 C mixtures.
- The 20 C mixture not only shows a good performance in term of stability and resilient modulus, but also indicates lower amount of asphalt consumed which comparable to granite control (GC) mixture.
- The use of ceramic waste as aggregates in asphalt mixture is showed an acceptable value in stability and resilient modulus. Regardless of comparing with the control sample, the value of both stability and resilient for all proportions were satisfied when stability and resilient modulus value obtained was more than 9.0 kN and 3000 MPa, respectively
- The incorporation of ceramic waste aggregate into HMA mixtures shows an acceptable. It proved that ceramic waste aggregates have potential to be used in industry for pavement construction and can acquire the benefit in terms of environmental and cost efficiency.

5. REFERENCES

KEYWORDS:
Bike Lane, Bike Lane Design, Urban Acupuncture, Urban Mobility, Urbanism

ABSTRACT:

On 2009 – 2010, after the needs of decent public space and creativity nodes emerged, the bicycling communities have also surfaced. Bike-To-Work and Bike-To-School communities have come out and becoming something’s trendy, especially amongst the youngsters. Before long, the bicyclists demand an ample road space to bike, which was considerably understandable.

Bandung City’s local government then decided to facilitate the need by establishing bike lanes in several routes, i.e. from Gedung Sate to City Hall and from Gedung Sate to Simpang Dago Area, which are considered as two potential routes for the pilot project’s establishment.

The bike lane establishment was also meant to reduce traffic congestion within the city by encouraging the citizen to use bicycle instead of motorized vehicles. However, the intention was facing its main problem, insufficiency of road dimension. The existing roads within old-Bandung-City area, where the routes are, are usually narrow because it was designed to be connecting roads within a garden city area. In contrary, the sidewalks and green strips along the street are relatively wide.

The bike lane design was implemented straightforward on 2010. The challenge has become real because it was dealing with existing physical problems, such as uneven street level and potholed asphalt / paving blocks, since the application of painting materials required a clean and even surface.

In the long run, since the number of bicyclist doesn’t increase significantly afterwards (compared to motorized vehicles growth, especially the motorcycles), the on-street bike lane was now in severe condition. The lanes were left neglected, without appropriate maintenance efforts.

Despite its unsuccessful effort in directly coping with traffic congestion, this bike lane establishment has raised the citizen’s awareness of reducing the use of motorized vehicles. Several new bicycling movements in Bandung have surfaced, including bike sharing and bike renting communities within Bandung City area.
Case Study in Urban Acupuncture: Bike Lane in Bandung City

Putrikinasih R. Santoso & Sigit Wisnuadji

KFA Studio, Bandung, Indonesia
Email for correspondence: sigwis@gmail.com

1 INTRODUCTION

In the year of 2009, bicycling movement has emerged and brought bicycle to a popular notion, especially among local young adults. It was introduced as an urban green-living lifestyle to the citizen of major cities in Indonesia, which then has made bicycling to be considered as a new fashion and social statement. It was also campaigned to be one of the alternative solutions in traffic congestion management within urban area. This positive promotion on bicycling triggered the raise of bicycling communities in big cities, such as Jakarta, Bandung, Bali, etc. The pioneer of bicycling communities in Indonesia is the Bike to Work Indonesia (B2W) Communities, who started their campaign on 2004. Later on, as the movement started to become recognized, the number of bicyclist within big cities increased accordingly. Before long, the bicyclists demanded an ample and decent bicycling route.

As one of the major cities in Indonesia, Bandung was affected by the trend consecutively. Despite Bandung’s hilly terrain, many see bicycling as an enjoyable way to travel within the city, especially the old Bandung area which was designed according to the Garden City concept (introduced by Ebenezer Howard on the late 19th century). Furthermore, bicycle is not a new mean of transportation in Bandung either. Bicycle was mainly used as the main transportation mode before motorized vehicles were introduced. Even during the Dutch colonialism era on early 20th century, on the era of the independence of The Republic of Indonesia in 1945, and even years after, bicycle is one of the most distinguished vehicles of all times. Only after 1960s, as the transportation technology grew, the bicycle started to be left behind.

Today, Bandung is a city of nearly 3 million inhabitants residing within approximately 17,000 hectares area (BPS Kota Bandung, 2014). Recent study from Bandung’s Society for Heritage Conservation shows that as the city expanded to neighboring suburban, Bandung City’s area has grown to nearly 600 % in the last eighty years. As Sandi A. Siregar stated on his dissertation, Bandung City development came in several stages, from its initial establishment along with Groote Postweg construction, its early development where the railroad and other urban facilities were built, to initiatives towards a capital city. This staged development left an interesting superimposed roads patterns within Bandung City area (Siregar 1990).

Figure 1. Comparison of the Current Bandung City Area and the Bandung Historical Area
(Source: Bandung’s Society for Heritage Conservation, 2014)
The city has been developing into a large conurbation, merging itself with neighboring area such as Cimahi, Soreang, and Ujungberung. Then again, this development was not without impact. Many people from Bandung City and Bandung Metropolitan Area still see the old city districts as the city’s center of attractions and activities. The commuters entering the city during the weekday can reach the number of 2,000,000 people each day, and even more on weekends. Severe traffic congestion is inevitable. In addition, Pasupati Flyover and Cipularang Highway establishment on 2005 has increased the ease of access to Bandung City. There are roughly 76,000 cars entering Bandung City from five different highway gates (i.e. Pasteur, Pasir Koja, Buah Batu, Moh. Toha, Cileunyi) each weekday and almost doubled the number on weekends. The city roads are nearly unable to bear the traffic burden nowadays.

As happened in any other major cities, the rise of bicycle movement gave a sheer expectation on dealing with the traffic congestion by promoting bicycle as an alternative transportation mode. In Bali, bike lanes connecting main tourist attractions in Kuta and Denpasar have been established. Later on, Bandung City’s local government decided to facilitate the need of bicycling route by establishing bike lanes in several routes as well. The effort was undertaken by Dinas Bina Marga dan Pengairan Kota Bandung under Vice Mayor’s direction. The first route was built on the west side of Jalan Ir. H. Juanda’s sidewalk. After completion of the first bike lane, Dinas Bina Marga dan Pengairan Kota Bandung leads an evaluation process on the bike lane with the help of local planner and architect. The evaluation process included a public engagement through community contribution in resolving next bicycling routes in Bandung City.

This case study aimed to reflect on the technical review of Bandung City Bike Lane as well as to expose the Bike Lane’s significance to Bandung City. The technical review will depict the ideal design process and framework, while the exposure of Bike Lane’s significance will give an understanding of the influences and side-effects on Bike Lane establishment in Bandung.

2 BIKE LANE IN BANDUNG

The pilot project for first Bandung’s Bike Lane is along the west sidewalk of Ir. H. Djuanda (Dago) street, which was designed as a shared lane on pedestrian area. The following bike lane establishment project took route along the Gedung Sate - Simpang Dago route (via Diponegoro, Sulanjana, and Ir. H. Djuanda streets) because the route connects Bandung City’s most notable governmental building and landmark (Gedung Sate) to more public areas in Dago area (factory outlets, high school, hospital, banks, and universities). According to the analysis conducted during the design process, there are several significant problems and challenges in designing a proper bike lane in Bandung, including but not limited to the narrow Right of Way (ROW) especially those within the “Old Bandung City” area; the high intensity of traffic in selected routes; the excessive number of hindrance along the sidewalk of selected routes; also the lack of drivers’ awareness to other slower vehicles and pedestrians. Shown on following image is the illustration of average condition of each section of the routes, i.e. Diponegoro Street, Ir. H. Djuanda Street. The addition of bike lane, whether it is on the street or on the sidewalks, will likely to raise conflict among the bicyclist and pedestrian or the bicyclist and motorized vehicle drivers.

Figure 2. Design Analysis on the Existing Road Dimension and Performance
(Source: KFA Studio, Bandung, Indonesia)
Taking the existing circumstances into account and consideration, thus designed a new bike lane on the selected route. The design recommendation consists of three main issues, i.e. to create a shared lane for bicycle and motorized vehicles on Diponegoro, Sulanjana, and some part of Ir. H. Djuanda Street, to create a dedicated lane on the sidewalks of most of Ir. H. Djuanda Street, and to create a safe intersection design. The implemented bike lane was nonetheless a dedicated lane on the street, or on the sidewalk, depends on the existing road performance (available ROW, traffic intensity, etc). Optimizing the available space, the bike lane was designed as a 1.20 m lane on each side of the road to be able to be used by a single bicyclist. Inevitably, bike lane was then interfered the traffic by narrowing the road width from 5.5 – 7.5 m to 4.3 – 6.3 m on each side. Below are the plan and sections of designed area as well as the illustration of the expected implemented design. On Figure 4 shown the proposed bike lane design along Diponegoro and Ir. H. Juanda Street, and on Figure 5 shown the proposed bike lane design specifically for the Cikapayang Intersection.

Figure 3. The Proposed Bike Lane Design Along Diponegoro Street (1, 2, 3) and Ir. H. Juanda Street (5, 6, 7)
(Source: KFA Studio, Bandung, Indonesia)

Figure 4. The Proposed Bike Lane Design on Cikapayang Intersection
(Source: KFA Studio, Bandung, Indonesia)
3 DESIGN IMPLEMENTATION AND IMPACTS

The bike lane design was implemented on 2010. This effort was, however, not without difficulties. Numbers of both expected and unexpected challenges have risen due to the existing physical condition of the road, such as uneven street level, potholed asphalt, etc. By the municipality’s request, the bike lane was painted in blue. Obviously, there are no logical reasons behind this decision. To support this request, it was proposed to use the thermoplastic paints for bike lane markings and colorings for the sake of durability. Unfortunately, the budget was reduced and the painting materials used for marking was changed to the regular outdoor paint, which caused them to away in a couple of weeks. Figure 7 below shows how the post-establishment bike lane condition on 2010. Its condition two years later (2012) can be seen in Figure 8.

![Figure 5. The Implemented Bike lane](image)

Left: A Dedicated Lane on the Street; right: A dedicated Lane on the Sidewalk
(Source: Santoso, 2010)

![Figure 6. Condition of the Neglected Bike Lane After Two Years of Implementation](image)

(Source: Santoso, 2012)
In the long run, the bike lane has failed to sustain. Many see it as a mere impulsive effort in response to current happening trend: the bicycling movements. There was no significant increment of bicyclist within Bandung City area following the establishment of this bike lane – or a considerably minor increase – compared to the upsurge of motorized vehicles. Later, the lanes were left neglected, without appropriate maintenance efforts. However, although its impact on Bandung City’s urban mobility in general remained undetermined yet, the bike lane establishment has raised more awareness of reducing the use of motorized vehicles. Several new bicycling movements in Bandung have surfaced, including bike sharing and bike renting communities within Bandung City area (see Table 1 below, the highlighted rows are the communities who started their activities after the establishment of bike lane in Bandung City). The communities remain active and improving their activities until today. The exact number of their active members, however, is hardly to say due to the come-and-go and cross-community nature of their members. The communities mainly based their activity to weekly (or monthly) fun “bicycling-together” programs. Nevertheless, Bike Sharing (or Bike .Bdg) currently focuses their activity in research regarding bicycling and its possibility of coping Bandung City’s traffic congestion as well.

Table 1 Several Renowned Bicycling Communities in Bandung

<table>
<thead>
<tr>
<th>Bicycling Community</th>
<th>Establishment</th>
<th>Member</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Year</td>
<td>After Bike Lane</td>
</tr>
<tr>
<td>1. Jarambah</td>
<td>2006</td>
<td>N</td>
</tr>
<tr>
<td>2. Koskas Bandung</td>
<td>2013</td>
<td>Y</td>
</tr>
<tr>
<td>3. Sel-B</td>
<td>2010</td>
<td>N</td>
</tr>
<tr>
<td>4. Bike to Campus Bandung</td>
<td>2013</td>
<td>Y</td>
</tr>
<tr>
<td>5. Bike to School Bandung</td>
<td>2012</td>
<td>Y</td>
</tr>
<tr>
<td>6. Sepeda Campus Bandung</td>
<td>2010</td>
<td>Y</td>
</tr>
<tr>
<td>7. BDGBMX</td>
<td>2010</td>
<td>N</td>
</tr>
<tr>
<td>8. Fixed Gear Bandung</td>
<td>2011</td>
<td>Y</td>
</tr>
<tr>
<td>9. Paguyuban Sapedah Baheula Bandung</td>
<td>2004</td>
<td>N</td>
</tr>
<tr>
<td>10. Paguyuban Sapedah Onthel Bandung</td>
<td>2012</td>
<td>Y</td>
</tr>
<tr>
<td>11. Bike Bandung Trial Community</td>
<td>2010</td>
<td>N</td>
</tr>
<tr>
<td>12. Lowrider Bandung BICC</td>
<td>2010</td>
<td>Y</td>
</tr>
<tr>
<td>13. Bandung Cycling Chic</td>
<td>2009</td>
<td>N</td>
</tr>
<tr>
<td>14. Bike Sharing (Bike.bdg)</td>
<td>2012</td>
<td>Y</td>
</tr>
</tbody>
</table>


4 BIKE LANE IN BANDUNG: A TRIAL WITH ROOMS FOR IMPROVEMENT

Bicycle often considered as one of the most effective and efficient transportation mode in terms of space requirement and discoverable distant, especially within the most city center area where the terrain is relatively plain and the significant obstacles and hindrances is at minimum level. It has become the notable alternative transportation mean especially in developed countries worldwide. The rise of bicycling movement in Indonesia on ca. 2010 has become a potential milestone to urban mobility in major cities like Jakarta, Bali, and Bandung because it triggered young people’s awareness on the possibilities and abilities of non-motorized vehicle for an easy affordable in-the-city transportation mode. The trend, however, should be evaluated and analyzed carefully so that it would share a positive benefit for the city.

Many of the proposed infrastructure designs were unimplemented (or failed miserably upon its completion) due to the misguided planning. A thorough and comprehensive planning on integrated urban mobility system may improve the quality of designed bike lane because it may offer a wider range of transportation mode choices in combination to bicycle, enhance its discoverable distant within the city. Thus, to generate a thorough bike lane master plan, it is obligatory to perform a thorough analysis on the exact transportation / mobility problem as well as the basic solution required to cope with the challenge. In addition, analysis on possibilities of multiple transportation mode combination should be conducted as well, e.g. combination of bicycle – bus – bicycle, or bicycle – MRT – bus – bicycle. Therefore, a concise master plan of urban mobility should be generated at the first place to manage the whole mobility issues within the city. For instance, to cope with the traffic congestion in Bandung a deep understanding of current mobility pattern within Bandung City and Bandung Metropolitan area is required. Bandung, as many of the cities and conurbation in Indonesia, has developed in sprawling manner, extending its development to the periphery and neighboring areas, where the suburban areas naturally developed into residential areas surrounding the city (see Figure 9 and 10 below). Traffic congestion on the main roads is mainly caused by the movement of people from residential
area to the city center. It is worsened by the lack of access to break the density of residential blocks, especially those in the eastern, western, and southern part of the city. Figure 9 below shows Bandung City’s urban density per district per year 2010. Meanwhile on Figure 10 is shown that Bandung’s city center is surrounded by a large portion of housing areas, where the inhabitants work, study, and play in the city center. This condition has become one of the main causes of severe traffic congestion in Bandung City for years. The only way to go from the eastern part of the city to the west is only by taking vehicles; in this case, private vehicle always the fastest way to get to another place, therefore most of the inhabitants use private vehicles (cars and motorcycles) to travel within the city.

Figure 7. Bandung City Urban Density Map  
(Source: KFA Studio, 2011)

Figure 8. Illustration on Urban Morphology of Bandung  
(Source: KFA Studio, 2014)
In order to break the urban density and to increase ease of access between the eastern and western part of the city, it is important to establish a sufficient east – west access road. This access road doesn’t necessarily to be accessed by the motorized vehicle but may be an establishment of bicycle and pedestrian access because it can make use of the existing infrastructure. Therefore, no large new infrastructure establishment required. The red line on Figure 11 below is one of the possible East – West access for bicycle and pedestrian, connecting the eastern residential area, the city center, and the western residential area. With shelters and pit stops along the routes, it is possible to bicycling from eastern to western side of the city by minimum intervention to existing through traffic.

![Figure 9. Possible East – West Access for Bicycle and Pedestrian](source: KFA Studio, 2014)

With a thorough, innovative, and integrated planning, the bicycling movement may be transformed into an urban lifestyle and contributes positively in traffic congestion control within Bandung City area. Other than that, a public education is also required to spread the awareness and the understanding of the benefits of bicycling for the city. By doing so, it is possible to upsurge and endorse bicycle as the main transportation mode in the city, especially for younger people.

5 REFERENCES


http://b2w-indonesia.or.id/


http://www.infobdg.com/v2/komunitas-sepeda-di-bandung/


Accessibility and Mobility Improvement Through Skywalk And The Arrangement Of Pedestrian Network System

N. Tanan¹, Ridwan Kamil², Nazib Faizai³, Gatot Sukmara⁴, Pantja Dharma Oetojo⁵, Samsi Gunarta⁶, Iskandar Zulkarnain⁷, Didi Ruswandi⁸, Sandy Suhendar⁹, Lisa Surya Lestari¹⁰, Melky Koswara¹¹, Agus Hidayat¹², Sigit Wisnuadji¹³, Indra¹⁴, Pierre Mandin¹⁵, Soukaina Legdani¹⁶

¹,³,⁴,⁵,⁶Institute of Road Engineering, Ministry of Public Works, Jl. A.H. Nasution 264 Bandung, 40294 Indonesia. ¹E-mail: natalia.tanan@pusjatan.pu.go.id
²,⁷,⁸,⁹,¹⁰,¹¹Municipality of Bandung, Jl. Wastukancana No. 2 Bandung, 40000 Indonesia
¹³,¹⁴Consultant, Bandung Indonesia
¹⁵,¹⁶Ecole Nationale des Travaux Publics de l’Etat, France

Abstract
The municipality of Bandung has creative ideas in order to improve the accessibility and mobility of pedestrians and non-motorized transport users through the provision of skywalk and pedestrian network arrangement. Cihampelas road and Tamansari road are two corridors selected as a strategic area. Non-motorized potential user in these regions are relatively large considering the characteristics of the area as a shopping and educational area. This paper identified a number of factors which support the selection of skywalk trace and the development of pedestrian network. It is expected to improve accessibility and mobility by encouraging people to walk especially for short distance trip. To build a sustainable pedestrian network and skywalk system, more attention should be made to the integrated parking area and public transit system, providing a pedestrian-friendly environment, and enhancing the urban image and landscapes

Keywords: accessibility, mobility, skywalk, bandung, pedestrian network, non-motorized transport

1. Introduction
Construction of skywalk and the arrangement of pedestrian network in Cihampelas and Tamansari area is a creative idea of Bandung municipality in order to provide accessibility and improve mobility for pedestrians and non-motorized users by considering aspects of security, safety, comfort, integration, as well as equity. Cihampelas area would be familiar, good for the community as well as visitors from outside Bandung city. As a shopping area, Cihampelas presents ranging from retail, fashion, and souvenirs of Bandung. Tamansari area has three special functions, i.e: education, industrial, and residential. A number of area in the region were maintained as green open space and local protected areas along the riverbanks of Cikapundung.

In addition, this area has a city landmark, Pasupati flyover, which connects the east and west of the city. From this area we can also enjoy the view of Mt. Tangkuban Perahu. Potential movement of the non-motorized user is relatively large considering the characteristic of these areas as a shopping and educational area. Therefore, the providing of pedestrian facility is required. The average distance from the residence to the education area just around 15-20 minutes which is ideal for walking distance.

This paper is focused on the development of the concept of connectivity region into the skywalk construction and pedestrian network. This concept becomes input for skywalk planning which considering the aspect of ease of movement, integration mode, accessibility, location of parking, as well as optimizing tourist attraction and leisure. Minimization of the environmental and social impacts is also getting attention. The aim of this paper is to identified a number of factors which support the selection of skywalk trace and the arrangement of pedestrian network.
2. Literature Review

2.1 Pedestrian Network

A Principal Pedestrian Network is a designated network of routes in a given area which support walking trips into and around key destinations such as activity centres, schools and transport nodes (Department of Transport, Planning and Local Infrastructure Victoria, 2013). Pedestrian access is integral to the functioning of a city and an important element of the transport system for the many short trips people make, including trips to public transport stops.

2.2 Skywalk System

Skywalk system refers to a network of elevated walkways that link individual skyways and buildings. Skywalk systems are usually linked to retail space, professional offices, and department stores in a city core. The idea of allocating pedestrians and vehicles into different layers was first proposed by Europeans (Hass-Klau, 1990). But the concept has mostly been realized in North America (Robertson, 1994). The first skywalk was opened in Minneapolis on 1962, which became a local scenic spot and caused the increasing of property values of those second-story retailers and had attracted large numbers of people to the area (Kaufman, 1985). The advantage of skywalk system are as follows:

- Creating Pedestrian Friendly Environment
  Skywalk can improve pedestrian safety by separate pedestrians from vehicular traffic (Robertson, 1994). It can also create a vibrant environment if it functions as well as the path described in Image of the City (Lynch, 1960), which provides vantage points for pedestrians to enjoy different views of the city.

- Encouraging Commercial Development trough traffic and economic aspects
  People have changed their travel behaviors because of the development of skyways in downtown. The skyway system allows people to park in the parking garages located on the fringes of the downtown area and then circulate through the system to reach their destinations in the city core. Such a comfortable, safe, and interesting pedestrian space therefore attracts the public to constantly return to the downtown (Robertson, 1994). Skywalk development not only intended to enhance the pedestrian space but also hope skywalk can attract people from the edge to the center of town and therefore revitalizes the downtown area (Kaufman, 1985) to argue that it is able to increase the economic value of a multilevel city.

3. Development Plan

3.1 Bandung City Government’s Policies Related to the Arrangement of Pedestrian Network

Bandung city government has established a number of programs related to the provision of a pedestrian network, as follows:

- developed a network of pedestrian facilities that already exist;
- build a network of pedestrian facilities at the required location;
- develop integrated pedestrian network facilities with centers of activities; and
- Build and develop facilities for pedestrians crossing

3.2 Existing Condition and Potential Development of Pedestrian networks in the study area

3.2.1 Existing Condition of Tamansari Area

The main problem in Tamansari area related to land use was residential development on the river banks of the Cikapundung due to the limited land for housing, while the population is increasing. This poses a threat to the transfer function of riverbank green line into settlements.
3.2.2 Existing Condition of Cihampelas Area

Because of the strategic location and high intensity commercial activities as well as services, encourage this area become the centers of economic activity in the city of Bandung. The development of Cihampelas area as a center of trade as well as services in Bandung cause great traction and lead to an increase in the traffic volume. Cihampelas road is a secondary collector, linking the northern and central parts of the city of Bandung. The high volume of traffic movement through this section can not be accommodated by the existing road network, resulting in an imbalance between supply and demand on the transportation.

The condition ultimately lead to the transport problems (congestion and the high vehicle-pedestrian conflicts) due to: side friction (on-street parking, trees in the carriageway, pedestrians walking on the road, etc), the occupancy of sidewalks by street vendors and parking so that pedestrians are forced to walk on the road.
3.3 Skywalk Development and Pedestrian Network Arrangement

Bandung skywalk development is expected to facilitate community activities without being disturbed by traffic congestion. In addition it is expected to reduce the use of motor vehicles. The current congestion on the Cihampelas and Tamansari road because of the high volume of vehicles on both sections. The development of the skywalk will be synergized with the arrangement of pedestrian network in these area as well as the provision of parking at the site: Gelap Nyawang, Tamansari, and Sabuga. Provision of parking space is expected to eliminate on-street parking. The visitors/tourists are expected to park in the parking space and then continue activities by walking in the city. Pedestrian network development plans can be seen in Figure 4.

![Pedestrian Network Plan in Tamansari – Cihampelas Area](image_url)

Figure 4. Pedestrian Network Plan in Tamansari – Cihampelas Area

The planning of the pedestrian network arrangement can be either at-grade or separated lane. As for some alternative skywalk development in the region is as shown in Figure 4. The selection of skywalk option because the terrain conditions which do not allow build at-grade pedestrian lane.
From Figure 5 above, several alternative skywalk development are as follows:

- Alignment will be made from Cihampelas then cross Cikapundung River (trace 1) Phase 2 will be made from the end of the alignment stage 1 after crossing the river Cikapundung then passed over the Zoo and ended in in the Tamansari section (trace 2). Trace length 450m
- Trace attaching end of the Bandung Skywalk on Cikapundung side, go through parking space planning in Gelapnyawang (trace 3). Trace length 640m.
- Trace attaching end of the Bandung Skywalk on Cikapundung side, go through parking space planning in Tamansari (trace 4). Trace length 776m
- Trace attaching end of the Bandung Skywalk on Cikapundung side, go through parking space planning in Sabuga (trace 5). Trace length 916m

3.4 Development of Cikapundung’s Riverbanks

One of the potential pedestrian network arrangements in this region is along the riverbanks of Cikapundung. Structuring border Cikapundung River at the Tamansari area is concentrated to overcome the problems of riverbank that transformed into a settlement function, so that the general direction for structuring border Cikapundung River are as follows:

- Returns riverbank function by creating a tourism path along the banks of the river. It can be done by creating green space and pedestrian access along river banks so that the area can be a potential attractions as congregate zone and the public spaces of the city (Bandung City Regulation No. 6 of 2002)
- Make a development buffer in the form of inspection road that prevent the transfer function of riverbank into settlement, also serves to open access to the river for easy accessibility. So the pattern of achievement-oriented building to the river, changing the main access area where the entrance area begins from the river
- Building arrangement in Pulosari into public green space that serves as a central city environment and green space in the form of productive space (commercial) and attractions (shopping, pedestrian space and area information center)
This area has good potential for several reasons supporting in the region, among others:

- Potentially area for tourism, including traditional *kukuyaan* culture (potentially a white-water rafting)
- Strategic location (center of Bandung)
- Potential to be a transport lane (potential for mass transportation)
- Green space in the area along the river that border the river as a public space
- Strong supports from the municipality, the province and the center in order to revitalize area
- The existence of legal devices that support the land use and environmental conservation.

The plan of arrangement will be directed to the river corridor: create a tourism path along the banks of the river by creating green space as well as pedestrian access along the banks and integrated with residential blocks. Some concepts of the development of the Cikapundung corridor associated with pedestrian network arrangement are as follows:

- Creating a regional nodes of public spaces as a regional center and a city open space as productive space (commercial) and attractions (shopping, pedestrian space, information center)
- Create a building that is oriented to the river, changing the main access area where the entrance area begins from the river.
- Opening access to the river for easy accessibility especially to facilitate movement between units’ circulation towards the environment and public spaces (regional nodes). The development of a secondary collector roads which penetrate the region so that it becomes easy achievement.
- Area development oriented to pedestrian comfort. Pedestrian circulation pattern will be oriented in riverbank areas so that the space needs to be designed in order to create an exciting experience space for pedestrians.
- Creating space demarcation productive and comfortable with building a center of culture and social interaction.
- Development which oriented to pedestrian comfort, the circulation pattern will be oriented in riverbanks so that the space needs to be designed in order to create an attractive space for pedestrians.
- Creating a productive and convenient riverbanks through the construction of a cultural and social interaction center

4. **Development Strategies**

Some of the strategies that must be considered in the construction of the skywalk system and arrangement of pedestrian network:

a. **Connecting Public Transit to Skywalk and pedestrian network system**

A well-designed connection between public transit and the skywalk or pedestrian network system can effectively reduce the use of private vehicles and create a humanity-oriented urban environment.
b. Creating a Pedestrian-Friendly Environment
One major advantage of building skywalk and pedestrian network system is that it can provide a comfortable, climate-controlled pedestrian space. The planning department should therefore make sure that the system has already achieved its goal of creating a pedestrian-friendly environment and then adjust the design to enrich the diversity of the urban space.

c. Enhancing the Image of the Area
To create a vibrant and glamorous urban environment, the expected image of the area, the location of the skywalk and pedestrian network, and the possible visual impacts are the key elements in skywalk development strategies.

d. Setting up a Management and Maintenance System
To build a sustainable skywalk and pedestrian network system and effectively manage the system, establishing a management and maintenance system is critical point.

5. Conclusion
To build a sustainable pedestrian network and skywalk system, more attention should be made: integrated parking area and public transit system, providing a pedestrian-friendly environment, and enhancing the urban image and landscapes.

6. References
Department of Transport, Planning and Local Infrastructure (2013), Principal Pedestrian Network, Victoria
Bandung City Irrigation Department (2002), Bandung City Regulation No. 6 of 2002, the Implementation of Irrigation, Bandung Indonesia
Integrated Road Safety Management in Indonesia and The Role of the Indonesian National Traffic Police Corps IRC, Bali, Indonesia 2014

By M. Naufal Yahya, Indonesia

Abstract

In accord with the UN Global Decade of Action 2011-2020, Indonesia is committed to reducing its traffic fatalities by 50% by the end of 2020. Traffic accidents in 2010 were officially estimated to result in an annual social cost of about 3.1% of the Indonesian Gross Domestic Product (GDP), rising to 3.7% of GDP in 2011 (i.e., ~AUD 29.8 Billion of a total GDP equivalent to AUD 805 Billion in 2011). With a rapid expansion of motorcycle purchases, which account for over 80% of road fatalities, annual social costs could approach some AUD 39 Billion or 4.6% of GDP. The Indonesian National Traffic Police Corps (Korps Lalu Lintas Polri, or Korklantas) has a central role in achieving this objective. Korklantas’ role is specified in Law 22 of 2009 relating to road traffic and transportation, and includes responsibilities for: road policing, traffic management and traffic enforcement; accident investigation; accident reporting and analysis; driver licensing; vehicle registration; and traffic education. Law 22/2009 provides the legislative framework for road safety activities, but the direction is provided by the National General Plan for Traffic and Road Transportation Safety (Rencana Umum Nasional Keselamatan Lalu Lintas dan Angkutan Jalan, or RUNK), which was released in 2011. The RUNK identifies five pillars on which to build road safety and traffic enforcement policies and actions: road safety management; safer roads; safer vehicles; safer road users; and, post crash care. To ensure that reliable and valid accident data are available, Korklantas has – with World Bank funding – developed a web-based accident investigation system (AIS). After piloting in Central Java during 2012, the AIS is available nationwide. Access to comprehensive, reliable and accurate road accident data makes it possible to identify the specific roads, vehicles and road users which need to be targeted with road safety and traffic enforcement interventions. Not only is the IRSMS being used as an accident investigation and policing tool, the system is able to be used by road safety stakeholders. The ability to access up-to-date accident data coupled with the need for Local, Provincial and National road safety interventions, the IRSMS will aid decision makers to develop evidence based strategies to reduce casualties and improve road safety in Indonesia.

Keywords

Indonesia, Road safety, Accident information system, Road safety strategy, ISO 39001, Traffic policing, Road safety management

1. Introduction
Road accidents\(^1\) are a very serious problem in Indonesia. In 2010, Police reported 31,234 road accident fatalities, equivalent to a rate of road fatalities per 100,000 people of 12.1. This is high compared to Singapore with 4.8 fatalities per 100,000 people, and Australia with 5.2 fatalities per 100,000 people. The preliminary data for 2011 indicate that 30,629 people were killed, 35,787 were seriously injured and 107,281 were slightly injured in 106,129 reported road accidents in Indonesia. Commentators consider that this is an underestimate as traffic accidents may not be reported, and data are inconsistent and difficult to verify: the Indonesia Infrastructure Initiative (IndII), for example, has suggested that about 40,000 people died as a consequence of traffic accidents in 2010 [1], suggestive of a level of underestimation of road trauma of 20-25%. Based on the current trend, it is estimated officially that 37,500 people could die on Indonesian roads in 2020 [2](see Figure 1). However, estimates up to 65,000 traffic fatalities per year have been projected for 2020 [3]. The Indonesian Traffic Police Corps commissioned the development of an improved accident database, using a web based accident information and analysis system not only to define the number of casualties and accidents but to capture the details needed to implement and monitor the effectiveness of evidence-based safety interventions.

As part of Indonesia’s commitment to the UN Decade of Action 2011-2020 program, an ambitious target has been set to reduce these numbers by 50% to less than 18,750 deaths by the end of 2020 [4]. A five-year program of action was established to support road safety; this program ran over 2008–2012. Priorities have been established with external financial assistance from the World Bank, the AusAID-funded Indonesia Infrastructure Initiative (IndII), the Asian Development Bank, and other stakeholders. A particular priority is for road safety partnership actions among stakeholders to improve capacity by strengthening coordination and management of road safety. Developing capacity is a pressing issue, as the responsibility for road safety action has been, until recently, quite diffuse [5]. These priority programs include:

- Study of locations with a high occurrence of accidents (“blackspots”) to better inform decisions regarding road safety engineering and traffic enforcement programs;
- Improvement in the quality of traffic accident investigations and improvement of the traffic accident data recording system;
- Improvement in traffic education from an early age and improvement of the system for issuing driver licenses;
- Trials of a number of new traffic policing actions, including speed enforcement using electronic devices such as radar and LIDAR, and enforcement of drunk driving and drug driving.

These programs are most likely to succeed if they use measurable objectives, if all stakeholders are committed and play an active role in implementation, and if they are regularly reviewed to evaluate program success and apply any necessary changes in anticipation of new trends. A forecast impact of these activities on fatality reductions from traffic accidents is shown in the lower part of Figure 1; Year 2010 is used as the base year for the projections [2].

Improvement in the quality of accident data is urgent, as these data form the basis of safety program planning by all stakeholders and serve as performance indicators to assess road transport safety. The success of the UN Decade of Action 2011-2020 programme in Indonesia depends on accurate evaluations of various interventions, and these in turn depend on whether

---

\(^1\) In Indonesia, “accident” is used instead of “crash”. While not consistent with the Safe System approach espoused by Western nations, the Indonesian terminology is used in this paper.
accident data are recorded and reported accurately and systematically. Put simply, there is a need to establish an evidence base—statistical data, or practical facts—and to act to place that evidence—those facts—before decision makers, road users, and the general community.

The accident data for Indonesia are provided by the IRSMS Accident Information System (and related databases on driver licensing, vehicle registration, and eventually hospital attendance and insurance claims).

<table>
<thead>
<tr>
<th>Year</th>
<th>Population</th>
<th>Prediction</th>
<th>DoA Target</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2010</td>
<td>237,000,000</td>
<td>50,000,000</td>
<td>32,192</td>
</tr>
<tr>
<td>2011</td>
<td>237,521,400</td>
<td>52,500,000</td>
<td>32,687</td>
</tr>
<tr>
<td>2012</td>
<td>238,043,947</td>
<td>55,125,000</td>
<td>33,189</td>
</tr>
<tr>
<td>2013</td>
<td>238,567,644</td>
<td>57,881,250</td>
<td>33,698</td>
</tr>
<tr>
<td>2014</td>
<td>239,092,493</td>
<td>60,775,313</td>
<td>34,216</td>
</tr>
<tr>
<td>2015</td>
<td>239,618,496</td>
<td>63,814,078</td>
<td>34,742</td>
</tr>
<tr>
<td>2016</td>
<td>240,145,657</td>
<td>67,004,782</td>
<td>35,275</td>
</tr>
<tr>
<td>2017</td>
<td>240,673,977</td>
<td>70,355,021</td>
<td>35,817</td>
</tr>
<tr>
<td>2018</td>
<td>241,203,460</td>
<td>73,872,772</td>
<td>36,367</td>
</tr>
<tr>
<td>2019</td>
<td>241,734,108</td>
<td>77,566,411</td>
<td>36,926</td>
</tr>
<tr>
<td>2020</td>
<td>242,265,923</td>
<td>81,444,731</td>
<td>37,493</td>
</tr>
<tr>
<td>2025</td>
<td>244,942,599</td>
<td>103,946,409</td>
<td>40,462</td>
</tr>
<tr>
<td>2030</td>
<td>247,648,849</td>
<td>132,664,885</td>
<td>43,667</td>
</tr>
<tr>
<td>2035</td>
<td>250,384,999</td>
<td>169,317,747</td>
<td>47,125</td>
</tr>
</tbody>
</table>

Figure 1 (TOP) Predicted traffic fatalities in Indonesia 2010-2035 and the targeted reduction under the Decade of Action (DoA); (BOTTOM). Targeted reduction in fatalities in Indonesia under to the Decade of Action of Road Safety (from [2])
The IRSMS Accident Information System is central to understanding the road safety situation in Indonesia, as it specifies who was involved, what happened immediately prior to, during and after the accident; where the accident occurred; when the accident happened; and describes how the accident took place; and, through police investigations and witness accounts, can establish why the accident occurred.

Of course, there needs to be a belief that change can happen. The moral compass for traffic accident reduction and improvement to road safety in Indonesia is provided by the Safe System approach, as expressed through the strategic plan (RUNK 2011-2035) [5] and action plans developed to address and guide road safety and traffic policing efforts.

2. The Indonesian National Traffic Police Corps

The Indonesian National Traffic Police Corps (INTPC), Korps Lalu Lintas Polri (or Korlantas) is an independent policing agency under the Indonesian National Police. The INTPC recognises that there is an increase in road trauma across Indonesia, and thus there is an imperative for action to implement more effective traffic policing actions to address road safety risk areas. Institutional capability reviews of INTPC have indicated that the organisation is disciplined and led by experienced senior officers [3]. While there is a good training capability at the Police Academy in Semarang and at the Traffic Education Centre near Jakarta, operational traffic policing capability needs to be improved to both detect, contain and reduce illegal road behaviours and to change inappropriate or risky behaviours. In order to do so, further institutional development is required to improve the professional and operational capabilities within INTPC. This is already underway, with budgetary responsibilities being shifted from central to provincial (Polda) levels. It is proposed that a safety directorate be established within Korlantas, tasked with the operation of the IRSMS which includes the accident reporting system, traffic accident statistical analysis and reporting (with more than 3,000 possible analyses available), as well as with stakeholder liaison, audit and quality control functions, training in accident data collection, and a traffic technology development function [3].

3. Law 22 of 2009 on Road Traffic and Transportation

The legislative framework for road safety in Indonesia is primarily provided by Law 22 of 2009, relating to Road Traffic and Transportation. That is, the primary responsibility for road safety rests with the INTPC rather than with Indonesian transport or public works agencies, although these other agencies retain road safety structures. Under Law 22/2009, the INTPC is charged with the responsibility for road traffic and transport safety. Generally, Law 22/2009 (Article 4, 5 and 12) aims to develop and organize a secure, safe, orderly and smooth land transportation system through:

- The movement of vehicles, people and/or goods on roads;
- The use of traffic and road transportation infrastructure and facilities; and
- Activities related to registration and identification of motor vehicles and drivers, traffic education, traffic management, engineering, and the enforcement of traffic and road transportation laws.

More specifically, the INTPC is charged with:

- Testing applicants and controlling licences for driving motor vehicles;
- Motor vehicle registration and identification;
• Collection, monitoring, processing and presentation of traffic and road transportation data;
• Traffic regulation, surveillance, escorting and patrolling;
• Law enforcement including actions against violations and handling of traffic accidents;
• Traffic education;
• Implementation of traffic management and engineering; and
• Implementation of traffic operational management.

A number of additional laws apply to particular issues, (e.g., provisions of Law 27/2009 on narcotics, relating to taking samples of blood, urine, etc., are relevant to addressing drug driving).

Accident data – information about the circumstances of an accident – are the basis for all targeted road safety interventions. For example, access to comprehensive, reliable and accurate road accident data makes it possible to identify specific roads, vehicles and road users which need to be targeted with road safety interventions. Road safety data can also be disseminated to relevant stakeholders and can aid decision making about the overall direction and strategy for road safety in Indonesia. While road users are required to report accidents, such reports can be made to local INTPC officers up to 40 days afterwards. As well, Law 22/2009 allows accidents to be “resolved at scene”, that is, to be negotiated between the affected parties. This means that an unknown number of accidents may be unreported, as they are settled between the parties and may not be recorded within the accident information system.

4. The National General Plan for Traffic and Road Transportation Safety (RUNK)

The National General Plan for Traffic and Road Transportation Safety (Rencana Umum Nasional Keselamatan (RUNK) Jalan 2011-2035) [4] was released on 11 May 2011, and reflects the goals outlined for the UN Decade of Action for Road Safety. Indonesia organized national events to launch the RUNK, and used the opportunity to advocate for increased attention on the road safety issue. The RUNK estimates that traffic accidents result in an annual social cost estimated to be at least 3.7% of the Indonesian Gross Domestic Product (total GDP is approximately Rp. 831 Trillion, or AUD 805 Billion).

The RUNK has identified five pillars on which to build road safety and traffic enforcement policies, based directly upon the Global Decade of Action for road safety 2011-2020:

• Pillar 1 relates to Road Safety Management. There are number of activities envisaged and undertaken under this Pillar, including:
  • Establishment of Forum on Road Safety at executive government level - enacted in Law 22/2009;
  • New Government Regulations to regulate road security and road safety.
  • Inclusion of the Provincial and Regency/City Governments – all levels of government are to take an active role in road safety;
  • Targeting the business sector and civil society to take more responsibility for remedial measures to improve road safety, and to promote road safety information; and
• Bringing leaders in Indonesian society, such as imams and other religious leaders, into the campaign on road safety.

• Pillar 2 concerns Safer Roads. Specific program actions have been identified, including projects to provide:
  • Safer Roadways;
  • Safer Road Planning and Construction (including road furniture); and
  • Safer Road Environment

• Pillar 3 concerns Safer Vehicles. There are a number of activities envisaged under this Pillar, but an important aspect for vehicle occupant safety is:
  • Legislative reform is needed make the use of rear seat belts mandatory. The mandatory wearing of seat belts in the front seats of vehicles only has been applied in Indonesia since 1993 (with Law 14/1992, as later revised by Law 22/2009).

• Pillar 4 concerns Safer Road Users. There are a number of activities envisaged and undertaken under this Pillar, including
  • Indonesia Road Safety Week, initially at a limited number of provincial levels in 2010-11, but to be extended to all provinces and to regencies and cities over 2012–2020
  • Increasing Government Agency participation
  • Increasing public participation
  • Increasing corporate participation

• Finally, Pillar 5 relates to Post Crash Care. When accidents occur, and it is recognised that in the operation of the road transport system accidents will occur as road users are fallible and make mistakes, then the response and timing of the actions of police, emergency services, healthcare and insurers is important.

To date, most activities have dealt with Pillar 2, relating to Safer Roads [1, 6], but capacity development actions to address all Pillars have been undertaken under IRSMS [3] and under the Indonesian Transport Safety Assistance Program (ITASAP) by AusAID [7,8].

5. Safe System and the road transport system

The UN Decade of Action for Road Safety and the Indonesian Road Safety Master Plan are based on the belief that improvements in road safety and reductions in road trauma are possible, and that the greatest road safety gains into the future will be achieved through adopting a Safe System approach. The primary aim of the Safe System approach is to prevent accidents from happening, and, in the event of a crash, to ensure that the impact forces released are within the boundaries of human tolerance and that no fatalities or serious injuries resulting in life-long disability will occur [9, 10]. Currently, what a road user understands about the sensibility and appropriateness of a road rule and what they accept as being sufficiently "safe" for travel, are not what is desired by the general community. That is, drivers don't necessarily know or fully understand why a particular traffic law is in place ("what it's for"), and drivers can often have a misplaced faith or expectation that the road on which they are driving, their vehicle, and other drivers are all sufficient to provide a "safe" place, seemingly independent of the manner in which they themselves are driving. In the immediate to medium term, a focus on the management of occupant protection devices
(motorcycle helmets, seat belts), vehicles, the road infrastructure, and driving speeds will likely best minimize the probability of death or serious injury as a consequence of a road accident. Appropriate and well-designed behavioural countermeasures are desirable, but reliable mechanisms for ensuring that road users are always alert and attentive, and are compliant with traffic laws, are not well understood or well implemented at present [11].

In the widest view [12], it is accepted that the desires of the population in any jurisdiction, in road transport terms, can be expressed as:
- Wanting to be mobile, that is, to be able to travel;
- Wanting to have access to transport options (road, public transport);
- Wanting to be safe;
- Wanting to have a sustainable environment; and
- Wanting to have a “pleasant” environment in which to live (amenability)

The IRSMS project is built on these general concepts.

6. The concept of an Integrated Road Safety Management System

The Indonesian IRSMS has been independently developed as a practical system to address road trauma and improve road safety [13]. For this system, the following elements were considered to be necessary to underpin effective management, target setting, the development of countermeasures and interventions, and evaluation of actions taken, as shown in Table 1:
- Enactment of a legislative framework to regulate the road transport system
- Access to valid and reliable data (the practical facts) concerning road trauma and behaviour
- A belief that change can happen

Table 1: Elements considered necessary for an Integrated Road Safety Management System (from [3])

<table>
<thead>
<tr>
<th>A framework</th>
<th>The legislative framework for an Integrated Road Safety Management System in Indonesia is provided by Law 22 of 2009, relating to Road Traffic and Transportation, and related laws. Law 22/2009 establishes INTPC as the lead institution for road safety.</th>
</tr>
</thead>
<tbody>
<tr>
<td>The practical facts</td>
<td>The data on traffic accidents are provided by an Accident Information System that specifies who is involved; what happened immediately prior to, during and after the accident; where did the accident occur; when did the accident happen; how did the accident take place; and, why did the accident happen.</td>
</tr>
<tr>
<td>A belief that change can happen</td>
<td>The moral compass for traffic accident reduction and improvement to road safety is provided by the Safe System approach, as expressed through the strategic plan (RUNK 2011-2035) and action plans developed to address and guide road safety and traffic policing efforts.</td>
</tr>
</tbody>
</table>
7. ISO 39001 – Road traffic safety management system

ISO 39001:2012 “Road traffic safety (RTS) management systems – Requirements with guidance for use” was developed to support the United Nations' Decade of Action for Road Safety 2011-2020 and published in late 2012. The ISO 39001 standard sets out the minimum requirements for a Road Safety Management System [14, 15, 16].

The ISO 39001 standard is intended to be a practical tool for governments, vehicle fleet operators and all organizations worldwide who want to reduce death and serious injury associated with road accidents. The standard is intended to be a tool that can be used to support strategies and actions to address risk in the road transport system, including the setting of ambitious road casualty reduction targets, the documentation of performance relative to those targets, and the sharing of experiences. It was developed from ISO standards such as ISO 9001 for quality management, including the plan-do-check-act cycle, and a requirement for continual improvement by all public or private sector organizations involved in regulating, designing or operating road transport. It will also help by providing a framework for contracts and communication between regulators, vehicle manufacturers and their suppliers.

8. Background to the IRSMS project for a national road safety management system

The Strategic Roads Infrastructure Project (SRIP) is supported by a loan from the World Bank (IBRD Loan 4834–IND), and has been implemented by the Directorate General of Highways within the Ministry of Public Works since late 2007. Following Law 22/2009, INTPC took over the responsibility for developing IRSMS. Project implementation is expected to be completed by mid-2013. The technical assistance to IRSMS is being undertaken by Consia Consultants.

The SRIP Project included a Road Sector Institutional Development component consisting of:

- IRSMS-1, to develop an integrated Road Safety strategy and long-term plan, including an institutional framework; via the Directorate General of Land transport (DGLT), later cancelled, and
- IRSMS-2, to develop a pilot integrated road accident database/analysis system, and establishing self-sustaining personnel development procedures for the INTPC.

Within INTPC, the IRSMS Project has delivered the following key achievements:

- Development of a web-based accident information and analysis system with a simple user interface for reporting and retrieving accident information;
- A new Accident Record Form has been developed, and Tablets using open source Android operating systems are being procured to improve both data quality and input times;
- An AIS User Manual for data entry and basic reporting has been published;
- From 1 September 2012, accident data collection, coding, entry and processing in the IRSMS server has been extended to the whole of Indonesia, in total, 445 Polres (police districts) of 31 Poldas (provincial offices);
- Daily accident reports are available to the Police Operations Department;
- Presentations on the Accident Information System (AIS) and training on the use of the new system and Accident Record Form has been provided to more than 430 police officers from the 31 Poldas, as well as to stakeholders and to police officers.
undertaking executive training for senior positions; further training for 500 personnel is planned in the first half of 2013;

- Two workshops on stakeholders’ data system requirements have been held;
- Training courses in road safety interventions have been developed, incorporating:
  - Development of a Road Safety Data Collection Manual
  - Development of a Data Analysis and Applications Manual;
  - Procurement of equipment for INTPC use in speed enforcement, drink drive enforcement, drug driving enforcement and overweight vehicle enforcement;
  - Development of Standard Operating Procedures (SOPs) for traffic enforcement by INTPC;
  - Development of local road safety implementation plans for INTPC Polda (provincial offices) and Polres (police districts) to conduct targeted operations based on the evidence from the IRSMS accident information system and local stakeholder consultation;
  - A series of media campaigns is being made for release in early 2013, including television commercials, newspaper advertisements, billboards and internet media. The campaigns will focus on the key priority means of reducing casualties based on the evidence from the IRSMS accident information system, and follow and support the themes of police traffic enforcement;
  - An IRSMS public website (www.korlantas-irsms.info) has been established, with webpages in both Indonesian and English languages that explain the system and provide additional background information.

Continued institutional development, training and capacity development in the present project will be closely linked to the development of the IRSMS, both under the SRIP Project to mid-2013 and beyond. In particular, much attention will be devoted to address the technical and institutional causes for the underreporting of road accidents.

9. The IRSMS Accident Information System

IRSMS is designed to provide valid, reliable and verified data for road accidents in Indonesia [2, 3, 13]. Information about the circumstances of an accident is the basis for all targeted road safety interventions [16]. For example, access to comprehensive, reliable and accurate road accident data makes it possible to identify specific roads, vehicles and road users which need to be targeted with road safety interventions.

Through Law 22/2009, the INTPC is charged with the responsibility for accident data collection and investigation. A user manual was developed to explain the methods and procedures that the INTPC needs to use to collect and analyse these accident data. The user manual provides basic and practical guidance for police and other stakeholders when entering accident data and utilising the information that is contained within the database system. At present, there are published versions of the user manual in both Indonesian and English. The AIS User Manual Version 1.2 describes the accident input process reporting a road accident under the IRSMS Accident Investigation System [17]. Further development for system users will address issues of data verification (validation), general data analysis and reporting, useability issues, and administration of the Accident Investigation System.

As well, an expansion has been approved that allows for a broadening of the scope of the project in two pilot provinces to include electronic data collection for accident reporting,
system design automation and digital transmission using tablet computers on site to gain automatic GPS location of the accident and to document the scene and gather relevant photographs and witness statements (if available).

10. System functionality

At present, data are collected by police filling in a notebook entry or the paper accident report form at the accident site. The information about the accident is then later entered onto the database at the police station. The location of the accident is registered as geographical coordinates, but this has occasionally been problematic as Indonesia straddles the Equator, and North/South latitude co-ordinates can be confused.

A user-friendly interface guides the registration of data from paper forms. The location of the accident can easily be corrected by simply dragging the accident indicator (see Figure 2) on a map. The name of the road is automatically registered once the accident location is selected and confirmed. Eventually, the data collection will be made by means of a tablet computer at the accident site, which will enable use of GPS for automatic registration of the location of the accident in geographical coordinates. Use of a tablet also enables data control to be effected at the accident site, minimising coding errors associated with multiple entry of data, as well as automatic transmission of data to the national database. Furthermore, photographic evidence and recordings of witness statements can be collected with the tablet and attached to the accident record. Images are stored as an integral part of accident information and can therefore be accessed at accident level, while witness statements are available to authorised users to support later criminal prosecutions. Additional documents can also be attached to the accident record. Output from the system is designed to serve for prosecution, investigation, planning and accident analysis purposes (for example, the system produces the main report that is necessary for court proceedings).

---

Figure 2: Screen for correcting the location of the accident
Figure 3: Zooming in on a map and selecting a specific accident
When analysing accident information, the easiest procedure is to use the map facility where accident concentrations can be found by zooming in on specific locations (see Figure 3). At any time the user can click on an accident and get a summary description of data and time, location, vehicles involved and injuries. An accident diagram and pictures of the accident are also displayed. A number of standard reports such as daily, weekly and monthly reports, and standard tables are also at the disposal of the user. On top of this the system, can generate a cross-tabulation of any given pair of variables in the system. Some cross-tabulations that are common to statistical reporting of accidents are provided as programmed options within the system.

11. Some final comments

Overall, IRSMS should improve the capacity of Indonesian agencies – and, in particular, the INTPC – to undertake actions to reduce road trauma and enhance road safety outcomes, thus further contributing to improvement of road safety in support of the Indonesian Road Safety Master Plan (RUNK) [4] and based on the UN Decade of Action for Road Safety.

IRSMS allows additional stakeholders (journalists, and the general public) to be able to identify specific data and to highlight where actions could be made, rather than providing accident data only to “approved” stakeholders from various government agencies, planners and road system designers. With IRSMS it is possible to give all stakeholders access to the web-based front end of an accident information and analysis system, as part of an integrated road management system as envisaged by ISO 39001. Using IRSMS, general accident data can be accessed by all stakeholders and still meet all legal privacy requirements. The confidentiality of individual names and personnel data is restricted to the needs of a particular registered user who can be allowed various levels of access (e.g., for the normal evidentiary data needs of police prosecution). The current data system used in Indonesia is simple enough to be accessed by any registered user anywhere globally, and more than 3,000 different summary report formats can be generated, as well as allowing for the investigation of the mechanics of individual accidents.

The design of the continued development of IRSMS will strengthen the institutional environment under which road safety and traffic policing actions are planned, undertaken and evaluated. If policies or decisions are based on limited or unreliable data, this can result in adverse results from program implementation and an unnecessary waste of resources. Road safety data, collected every day, can fulfill this purpose if they are properly recorded and compiled in a reliable system that can subsequently be used for data processing and analysis. The results can also be disseminated to relevant stakeholders and, when used effectively, can aid decision making on overall direction and strategy for road safety in Indonesia.

An analogy is that of “moving on an escalator of progress”. The “escalator” moves from a situation where doing any form of road safety activity seems limited and likely ineffective, to a vantage point from which activities can be initiated and expanded, where researchers can investigate real events, where journalists can access real data to report on what has worked and what has not worked, and where politicians in turn are able to be better informed and be more realistic in their major economic and safety decisions concerning funding allocations and infrastructure planning.
To summarise, one way of thinking about what is being done in Indonesia is a series of small iterative changes that introduce web-based access to a safety database that can not only be used by the hosts (the INTPC) but, once officially launched in April 2013, can expand rapidly to be used by all stakeholders in Indonesia. The aim of IRSMS, together with the RUNK and with planned legislative changes under Law 22/2009, is to provide the practical facts that can be used to accelerate institutional change and underpin decisions about capacity development needs. This should not only start to stabilise the rapidly increasing casualty rate associated with road accidents but to accelerate the reduction so that a 50% reduction by 2020 becomes a planned reality rather than a remote possibility.

12. Acknowledgements
I thank our leaders, that made this project possible and the Staff of the IRSMS-2 project. Consia Consultants from Denmark (C.Wass, I. Faulks, P. Hambleton, E. Rask & S. Wäesche).

References


Indonesian case study”. Paper presented at the 25th ICTCT Workshop, 8-9 November 2012, Hasselt, Belgium.


PAPER TITLE: Three-Cable Barrier System Adjacent to Steep Slopes

TRACK: Road Safety

AUTHOR: Ronald K. FALLER
POSITION: Director & Research Associate Professor
ORGANIZATION: Midwest Roadside Safety Facility Nebraska Transportation Center University of Nebraska-Lincoln 130 Whittier Research Center 2200 Vine Street Lincoln, Nebraska 68583-0853
COUNTRY: USA

CO-AUTHOR(S): Karla A. LECHTENBERG
POSITION: Research Associate Engineer
ORGANIZATION: Midwest Roadside Safety Facility Nebraska Transportation Center University of Nebraska-Lincoln 130 Whittier Research Center 2200 Vine Street Lincoln, Nebraska 68583-0853
COUNTRY: USA

CODY S. STOLLE
POSITION: Post-Doctoral Research Associate
ORGANIZATION: Midwest Roadside Safety Facility Nebraska Transportation Center University of Nebraska-Lincoln 130 Whittier Research Center 2200 Vine Street Lincoln, Nebraska 68583-0853
COUNTRY: USA

JOHN D. REID
POSITION: Professor
ORGANIZATION: Mechanical Engineering Department University of Nebraska-Lincoln W359 Nebraska Hall Lincoln, Nebraska 68588-0526
COUNTRY: USA

E-MAIL: rfaller1@unl.edu

KEYWORDS:
Highway Safety, Roadside Safety Features, Cable Barrier, Fill Slope, Cable Anchor, Crash Testing, and NCHRP 350

ABSTRACT:
Low-tension, three-cable barrier systems are often used to protect motorists from roadside slopes and placed in locations where relatively large lateral barrier displacements are acceptable. Concerns arise when these systems must be placed close to steep roadside slopes. As a result, the primary research objective was to investigate the crashworthiness of a common low-tension, cable barrier system when installed near steep fill slopes and modify the basic configuration, if needed. Since large reinforced concrete anchors are often used to terminate three-cable barriers, a secondary research objective was to configure, analyze, test, and evaluate several cost-effective end anchorage systems.

During the study, dynamic component testing was used to evaluate the capacity, performance, and effectiveness of three alternative cable anchorage systems to serve as a surrogate to existing, large reinforced concrete anchors. BARRIER VII computer simulation modeling was used to analyze and predict dynamic barrier performance with each anchor alternative.

One full-scale crash test was performed according to the National Cooperative Highway Research Program (NCHRP) Report No. 350 requirements using a 2,034-kg (4,481-lb) pickup truck impacting at a speed of 98.1 km/h (61.0 mph) and at an angle of 26.2 degrees. The full-scale crash test was conducted on a low-tension, three-cable barrier system placed adjacent to a 1%:1V fill slope with the weakest anchor alternative (i.e., driven steel post). Unfortunately, the crash test was unacceptable according to the Test Level 3 (TL-3) evaluation criteria due to barrier override and rollover. Consequently, design modifications were deemed necessary. BARRIER VII computer simulations demonstrated that decreased post spacing and increased lateral offset away from the slope break point significantly improved barrier performance.

The cable barrier system was modified to include quarter-post spacing and an increased lateral offset from the slope break point of 1.22 m (4 ft). The second TL-3 crash test was successfully performed according to the NCHRP Report No. 350 criteria with a 2,032-kg (4,481-lb) pickup truck impacting at a speed of 99.1 km/h (61.6 mph) and at an angle of 23.6 degrees. Therefore, a three-cable barrier system with steel posts positioned 1.22-m (4-ft) laterally away from the slope break point and spaced on 1.22-m (4-ft) centers was recommended for use in shielding steep fill slopes.
Three-Cable Barrier System Adjacent to Steep Slopes

Dr. Ronald K. Faller, P.E.  
Karla A. Lechtenberg  
Dr. Cody S. Stolle  
Dr. John D. Reid

1Midwest Roadside Safety Facility, University of Nebraska-Lincoln, Lincoln, Nebraska, USA  
2Mechanical Engineering Department, University of Nebraska-Lincoln, Lincoln, Nebraska, USA  
Email: rfaller1@unl.edu

1 INTRODUCTION

1.1 Problem Statement

Low-tension, three-cable guardrail was one of the first roadside barrier systems developed in the United States (U.S.). Over the years, three-cable barriers have been used to prevent errant motorists from impacting hazardous fixed objects and geometric features that are found along the roadside and within medians. These hazardous obstacles have included bridge piers, roadside trees, median ditches, uneven or rock-filled terrain, as well as steep slopes. In some scenarios, State Departments of Transportation have utilized low-tension, three-cable barriers to protect motorists from traversing fill slopes as steep as 1½H:1V. However, concerns have been raised regarding the safety performance of cable barriers when placed close to these steep slopes.

In general, cable barrier systems have incorporated steel cables that are mounted on widely-spaced, support posts. As a vehicle strikes a cable barrier, its steel cables are stretched, thus producing tension forces that contain and redirect an impacting vehicle. The steel cables are designed to crease and interlock into the sheet metal of an impacting vehicle. The support posts are either embedded in soil or inserted into socketed foundations. Under a crash event, the support posts are designed to bend, which limits accelerations imparted to an impacting vehicle. By design, three-cable barriers are generally categorized as flexible barriers due to their inherently large lateral displacements under vehicular impact loading. When considering the large lateral barrier displacements, three-cable barriers may have limited use in shielding motorists from hazardous obstacles located close to the edge of traveled way.

Even with the concerns noted above, there remains significant interest in using three-cable barriers to shield steep fill slopes. First, cable barrier systems are relatively easy to install and deemed economical. Material costs have traditionally been lower for cable guardrail as compared to most other barrier types. Second, cable elements have a lower projected area than provided by a single corrugated W-beam rail or a solid concrete barrier, thus reducing concerns for snow drifting nearby or actually on the roadway. Third, cable barriers are flexible and associated with decreased vehicle accelerations (i.e., a more gradual vehicle containment and redirection) as compared to impacts with semi-rigid and rigid barriers. Due to its increased barrier flexibility and greater length of deformed barrier, there exists a reduced risk for vehicles being rapidly projected back into nearby traffic in an uncontrolled manner and potentially causing serious secondary collisions. Thus, cable guardrail has been known for its excellent accident record, which has been attributed to the system’s low lateral stiffness and ability to limit secondary collisions (1-2).

Many research studies have been performed to investigate the safety performance of various cable barrier systems in varied applications. For example and in the late 1960’s, the Ontario Department of Highways sponsored a crash testing and evaluation program. This study concluded that cable spacing influenced a vehicle’s tendency to snag on guardrail components, and that guardrail deflection was directly related to the stiffness of the support posts (3). On another study, researchers at the New York State Department of Transportation (NYDOT) conducted several full-scale crash tests on cable barrier systems. The test results demonstrated that cable guardrail placed near 2H:1V roadside slopes can provide adequate containment and safety performance for large sedans (4-5). Even with the numerous cable barrier research studies beyond those noted herein, the safety performance of low-tension three-cable barrier placed near steep roadside slopes, such as 1½H:1V, had not been investigated under impacts with passenger vehicles, including those with light trucks.

Beyond crashworthiness concerns of low-tension, three-cable barrier installed near steep slopes, other issues have been raised. Highway agencies have reported the development of cable slack within the barrier over time, which generates a need for periodic system maintenance. Second, there had existed only one approved, non-proprietary guardrail end terminal for three-cable barrier, and it incorporated a flared end with a 1.22-m (4-ft) lateral offset with an extremely large, reinforced-concrete dead-man anchor system. When guardrails are placed adjacent to roadside slopes, this flared terminal end required significant grading in order to provide a relatively flat area surrounding the terminal. Third, the
large, reinforced-concrete anchor system often weighed more than 3,628 kg (8,000 lb). The cost associated with providing additional grading and a large reinforced-concrete dead-man anchor often exceeded the cost for installing the cable barrier system. If the flared configuration could be eliminated and/or the size and cost of the large concrete anchorage could be reduced, then three-cable barrier could become even more economical.

Based on the noted needs and concerns, the Midwest States Pooled Fund Program sponsored several research studies to investigate the safety performance of three-cable barriers installed near steep slopes and to configure alternative, cost-effective anchorage systems for terminating low-tension, three-cable barriers.

1.2 Research Objectives

The research objectives were to: (1) develop and evaluate alternative anchorages for terminating cable barriers that reduce installation and maintenance costs; (2) investigate and evaluate the safety performance of standard three-cable barrier subjected to light truck impacts when installed adjacent to steep roadside slopes, such as a 1½H:1V fill slope; and (3) implement design modifications to the cable barrier system, as needed. The three-cable barrier system was tested and evaluated according to the Test Level 3 (TL-3) safety performance criteria set forth in the National Cooperative Highway Research Program (NCHRP) Report No. 350, Recommended Procedures for the Safety of Performance Evaluation of Highway Features (6).

1.3 Research Plan

The research objectives were achieved through a series of tasks. First, a detailed literature review was conducted regarding the concept development, numerical modeling, and full-scale crash testing of cable barrier systems and the associated components, as described in References (7-8). Next, a dynamic component testing program was conducted on guardrail posts and alternative anchorage systems. Following the component testing program, computer simulation modeling with BARRIER VII was used to investigate dynamic performance and efficacy of various design alternatives placed within a cable barrier system (9). After selecting an alternative anchorage system, the cable guardrail system was fabricated and constructed at the Midwest Roadside Safety Facility’s (MwRSF’s) outdoor testing facility in Lincoln, Nebraska. Subsequently, one full-scale vehicle crash test was performed using a 2,000-kg (4,409-lb) pickup truck at the target impact conditions of 100.0 km/h (62.1 mph) and 25 degrees. Based on the test results, additional BARRIER VII computer simulation was performed to evaluate various design modifications that could improve the safety performance of three-cable barriers installed adjacent to steep roadside slopes. Following a redesign and continued simulation effort, a second full-scale vehicle crash test was performed using the same target impact conditions noted above. Further information and details pertaining to this research effort can be found in References (7-8,10).

2 GENERAL BARRIER CONFIGURATION

Common three-cable barrier systems utilize three separate wire rope elements that are attached to steel support posts, as shown in Figure 1. Each cable is fabricated as a 19-mm (¾-in.) high-strength galvanized wire rope, comprised of three 9.5-mm (3/8-in.) diameter strands with seven individual wires per strand. Although the rated strength for an individual cable has generally been specified at 111 kN (25 kips), test results have revealed an average breaking strength greater than 178 kN (40 kips) for material fabricated with improved plow steel (IPS) (11). The centerline height for the top cable has typically ranged between 686 and 762 mm (27 and 30 in.), while lower cables are incrementally spaced between 76 to 152 mm (3 to 6 in.).

Line posts are fabricated from rolled steel sections and include a bearing plate that is welded to the post in order to increase lateral soil bearing capacity. The line posts generally provide low weak-axis bending strength to allow posts to easily deform when struck in a direction parallel to the roadway. Cable hook bolts secure cables to posts, while permitting relatively free longitudinal cable movement along the system as well as vertical and lateral release away from posts when bent down during an impact event.

Spring-type compensating devices and turnbuckles accommodate the cable’s thermal expansion and contraction. When installed, the spring compensator is initially compressed to allow for temperature increases. Additional compressive stroke is available to accommodate extension when the temperature decreases. If the temperature decreases sufficiently, excessive cable tension can result in creep at the anchor system, thus creating slack when the temperatures rise again. Furthermore, large increases in temperature can also produce slack in the barrier system, which also can result in cable sag as well as a reduction in the barrier’s ability to engage an impacting vehicle.

At the upstream and downstream ends of a cable barrier system, massive reinforced concrete blocks are used to resist the cable tension loads, as shown in Figure 2. These anchors typically consist of cast-in-place or precast concrete elements, cast as either one or two units. These anchorage also provide a mounting location for the steel anchor plate to attach the cable ends as well as contain a slipbase assembly for one of the terminal posts. The steel anchor plate
allows cable release during reverse direction impacts near the terminal end. These large concrete blocks have volumes and masses ranging between 1.3 to 1.9 m$^3$ (1.75 to 2.5 yd$^3$) and from 3,175 to 4,535 kg (3.5 to 5.0 tons), respectively.
3 ALTERNATIVE ANCHORAGES AND DYNAMIC COMPONENT TESTING

As noted previously, the use of large, reinforced-concrete anchorage systems can result in significant installation costs for cable barrier systems. Thus, an effort was made to brainstorm, design, test, and evaluate three alternative anchorages to simplify construction of the low-tension, three-cable guardrail system. The derived alternatives included a reinforced-concrete block, a reinforced-concrete shaft, and a driven steel post. The alternative concrete block was similar to the existing design, but it was smaller in size and could more easily be lifted into place with existing equipment. The revised concrete block measured 1,524 mm long x 1,015 mm deep x 610 mm wide (60 in. x 40 in. x 24 in.) and weighed approximately 2,268 kg (5,000 lb). Additional details for the reduced-size block are provided in References (7-8). The second alternative consisted of a reinforced-concrete shaft that provides a simplified concrete alternative, but it still relies on the use of cast-in-place concrete. It consisted of a drilled shaft that was filled with reinforced concrete, which reduced the volume of concrete and simplified excavation. The drilled anchor shaft had a 457 mm (18 in.) diameter and was 1,829 mm (72 in.) deep. Additional details for this variation are provided in References (7-8). The third alternative incorporates a large steel beam that can be easily driven into the soil. The steel-post alternative effectively eliminates the need for concrete, utilizes readily-available equipment, and further simplifies the construction process. A W152x37.2 (W6x25) steel section with a 248-MPa (36-ksi) yield strength and a 2,438-mm (96-in.) overall length was selected, as shown in Figure 3. The 610-mm x 610-mm x 13-mm (24-in. x 24-in. x ½-in.) soil bearing plate provided additional anchorage to resist axial cable forces.

![Figure 3. Driven Steel-Post Anchorage](image)

It is commonly known that a single cable can capture and redirect high-energy impacts with passenger vehicles. As noted previously, some types of 3x7 wire ropes cables have demonstrated ultimate tensile capacities near 178 kN (40,000 lb). As such, one may initially and incorrectly assume that a cable anchorage system would need to resist a peak axial load of 534 kN (120,000 lb). However, this combined capacity was deemed unreasonably high and did not correlate with findings from prior field studies. Using knowledge gained from previous cable barrier tests in combination with the fact that a single cable can successfully capture high-energy impact events, it was believed that an anchor system could perform adequately if it could withstand a minimum tensile load of 178 kN (40,000 lb) over a reasonable displacement. A series of dynamic component tests were conducted with a surrogate vehicle (i.e., bogie vehicle) in order to verify that each alternative anchorage could sustain a 178-kN (40,000-lb) load with axial ground line deflections of 152 mm (6 in.) or less. The dynamic tests were performed with a 2,223-kg (4,900-lb) bogie vehicle tethered to each design. The tests were configured to determine anchor yield forces, peak load capacities, and associated displacements that could be expected during an impact event, as noted in References (7-8).

For this research study, dynamic bogie test results were summarized by force versus deflection behavior and are depicted graphically in References (7-8). These results were later used in a BARRIER VII computer simulation effort. The reinforced-concrete block provided the strongest alternative and is characterized by a 254-kN (57-kip) peak resistive force. The drilled shaft produced a maximum resistive force of 205 kN (46 kips), while the driven steel-post
anchorage proved to be the weakest option and sustained a 187-kN (42-kip) peak load. The results indicated that the minimum force and displacement criterion were only met by the concrete shaft and concrete block alternative anchorages. Although the steel-post anchorage was slightly below initial force and displacement criterion, it was believed to be a viable anchorage system and capable of sufficient axial loading. Based on these results, BARRIER VII computer simulation was utilized to investigate the cable system behavior with each prototype anchorage system.

4 BARRIER VII COMPUTER SIMULATION

BARRIER VII has proven to be a powerful tool for predicting barrier performance and has been used often for the analysis and design of longitudinal barriers, such as roadside cable barriers (9). The 2-D program is designed to analyze the dynamic behavior of a vehicle striking a deformable protective barrier, while accounting for large displacements and inelastic behavior. Finite element modeling is often utilized to evaluate and compare alternative concepts within the initial stages of design. BARRIER VII is normally calibrated against full-scale crash test results in order to gain maximum confidence in the analysis process. Additional details, simulation results, and significant discussion pertaining to the BARRIER VII model development, material parameters, and data inputs for barrier components, validations, and general level terrain simulations are provided in References (7-8).

One of the research objectives included an investigation of the dynamic impact behavior and adequacy of a three-cable barrier system installed adjacent to a 1½H:1V fill slope. Unfortunately, BARRIER VII is limited to two-dimensional (2-D) analysis and does not allow for 3-D modeling. Thus, the finite element model was modified to account for an impacting vehicle to strike the cable barrier and extend over the fill slope. As such, the additional energy produced by a vehicle traveling down the slope was added to the vehicle’s initial kinetic energy using Equation 1:

\[
\frac{1}{2}(m)(V_x \sin \theta)^2 = \frac{1}{2}(m)(V_n \sin \theta)^2 + mgh
\]

(Eq. 1)

where:
- \(m\) = vehicle mass
- \(g\) = acceleration due to gravity (9.807 m/s^2)
- \(h\) = vertical vehicle drop over slope associated with lateral barrier deflection on level terrain
- \(V_x\) = effective vehicle velocity with slope considerations
- \(V_n\) = vehicle velocity on level terrain per NCHRP Report No. 350 conditions
- \(\theta\) = impact angle (25 degrees)

For the BARRIER VII simulations performed on level terrain under NCHRP Report No. 350 impact conditions, the average lateral barrier deflections over several cases was approximately 4.0 m (13.0 ft). When considering a vehicle c.g. height of approximately 0.67 m (2.19 ft), a lateral barrier displacement of 4.0 m (13.0 ft), and a 1½H:1V fill slope, the vehicle’s vertical c.g. drop was estimated to be approximately 1.53 m (5.02 ft). Using Equation 1 with a vertical drop of 1.53 m (5.02 ft) and NCHRP Report No. 350 impact conditions, a modified impact velocity was determined in order to account for the potential energy attributed to a vehicle traveling down a slope and equaled approximately 110.4 km/h (68.6 mph).

The simulation results are briefly noted below but discussed in greater detail in References (7-8). The subsequent BARRIER VII simulation effort revealed an acceptable range of anchor displacements and anchor forces, a maximum lateral barrier deflection of approximately 4.19 m (13.75 ft), and a contact length of 41.45 m (136 ft). The large lateral barrier displacement raised significant concern as to whether an impacting vehicle could remain stable while extending over the fill slope. The 4.19-m (13.75-ft) simulated lateral deflection corresponded to an approximate 1.98-m (6.5-ft) vertical drop in the vehicle’s center of gravity (c.g.) as it encroached onto the slope. This significant drop greatly increases the potential of vehicle rollover or barrier override. Based on these results, full-scale vehicle crash testing was recommended to evaluate the safety performance of three-cable barrier installed near steep slopes.

5 TEST REQUIREMENTS AND EVALUATION CRITERIA

Longitudinal barriers, such as three-cable guardrail, must satisfy the requirements provided in NCHRP Report No. 350 to be accepted for use on new construction projects found along the National Highway System (NHS) or used as a replacement for existing designs that do not meet current safety standards. According to TL-3 of NCHRP Report No. 350, longitudinal barriers must be subjected to two full-scale vehicle crash tests. The two crash tests are as follows:

1. Test Designation No. 3-10 consists of an 820-kg (1,808-lb) small car [820C vehicle] impacting a barrier at a nominal speed and angle of 100.0 km/h (62.1 mph) and 20 degrees, respectively.
2. Test Designation No. 3-11 consists of a 2,000-kg (4,409-lb) pickup truck [2000P vehicle] impacting a barrier at a nominal speed and angle of 100.0 km/h (62.1 mph) and 25 degrees, respectively.
When considering the two impact conditions noted above, higher impact energy is associated with the 2000P pickup truck crash test as compared to the 820C small car crash test. As a result, larger barrier deflections and greater vehicle extent beyond the slope break point would be expected with the pickup truck crash test, thus increasing concerns for vehicle rollover, override, or penetration as compared to the small car test. Therefore, a 2,000-kg (4,409-lb) pickup truck test was deemed sufficient to evaluate the safety performance of a common three-cable barrier system placed in front of a steep fill slope, and an 820-kg (1,808-lb) small car test was considered unnecessary for this research study.

Evaluation criteria for full-scale vehicle crash testing are based in three appraisal areas: (1) structural adequacy; (2) occupant risk; and (3) vehicle trajectory after collision. Criteria for structural adequacy are intended to evaluate the ability of the barrier to contain, redirect, or allow controlled vehicle penetration in a predictable manner. Occupant risk evaluates the degree of hazard to occupants in the impacting vehicle. Vehicle trajectory after collision is a measure to evaluate the potential for the post-impact vehicle trajectory to cause subsequent multi-vehicle accidents and result in increased risk for the occupants of vehicles involved in secondary collisions. These three evaluation criteria are discussed in greater detail in NCHRP Report No. 350 as well as in References (7-8,10). Further, all full-scale vehicle crash tests were conducted and reported in accordance with the procedures provided in NCHRP Report No. 350 (6).

6 ORIGINAL BARRIER SYSTEM (0.3-m Lateral Offset – 4.88-m Post Spacing)

Based on the BARRIER VII simulation results, one full-scale crash test was planned to explore vehicle stability and evaluate barrier performance. The barrier system was based on a common three-cable guardrail and included the driven steel-post option, or weakest of the three alternative anchorages.

The cable barrier system was 147.82 m (485 ft) long and consisted of four major structural components: (1) wire rope cables; (2) steel support posts; (3) spring compensator end assemblies, threaded rod connectors, and couplers; and (4) end terminal hardware and foundation anchorages. These components are largely shown in Figures 4 through 6, while additional design details are provided in Reference (7-8).

Three 19-mm (¾-in.) diameter, 3x7 wire ropes were used for the rail elements, which were spaced 76 mm (3 in.) apart. The cables were positioned with centerline heights of 762 mm (30 in.), 686 mm (27 in.), and 610 mm (24 in.), which were supported by thirty-two posts spaced on 4.88 m (16 ft) centers. The cables were attached to line posts using three 8-mm (5/16-in.) diameter, standard cable hook bolts. The line posts were 1,600-mm (63-in.) long and fabricated from ASTM A36 S76x8.5 (S3x5.7) steel sections. The post sections incorporated a 203-mm x 610-mm x 6-mm (8-in. x 24-in. x ¼-in.) soil bearing plate that was welded to the backside flange and were embedded 762 mm (30 in.) into the soil. The guardrail posts were installed approximately 0.3 m (1 ft) away from the slope break point of a 1½H:1V fill slope.

The cables were tensioned using spring compensators, while the ends of the cables were fitted with threaded rods that terminated at the foundation anchorage systems. The threaded rods were attached to the end anchorage with three washers and two 19-mm (¾-in.) diameter Grade 5 nuts. As noted previously, existing reinforced concrete anchorages have also been used to support the first slipbase post on each end. Thus, a surrogate anchorage was used for the foundation of the first slipbase post on each end. Details for the modified foundation for the first slipbase post is contained in References (7-8).

One steel-post anchorage was used on each end and fabricated from an ASTM A36 W152x37.2 (W6x25) steel section and embedded in the soil to a depth of 2,438 mm (96 in.). A 13-mm (½-in.) thick by 610-mm (24-in.) square, ASTM A36 steel soil plate was welded to the post flange by a series of 10-mm (¾-in.) fillet welds. Furthermore, a steel cable anchor bracket was welded to a 13-mm x 356-mm x 248-mm (½-in. x 14-in. x 9¾-in.) steel plate that was welded to the top of the vertical anchor post.

A 64-m (210-ft) long x 10-m (33-ft) wide pit was excavated behind the cable barrier system. The front profile of the sloped section was 6.1 m (20 ft) wide by 4.0 m (13 ft) deep, thus creating a 1½H:1V fill slope.
Figure 4. Three-Cable Barrier System Adjacent to Fill Slope

Figure 5. Three-Cable Barrier System Adjacent to Fill Slope (continued)

Figure 6. Three-Cable Barrier System Adjacent to Fill Slope – Upstream End Anchorage

7 TEST NO. CS-1 (2,034 kg – 98.1 km/h – 26.2 deg)

The 2,034-kg (4,484-lb) pickup truck impacted the three-cable guardrail system at a speed of 98.1 km/h (61.0 mph) and at an angle of 26.2 degrees. The target impact point was 2,134 mm (84 in.) downstream from post no. 12, as shown in Figure 7. Actual vehicle impact occurred 2,743 mm (108 in.) downstream from post no. 12.
During the impact event, the vehicle became completely airborne and pitched downward, rolled counter-clockwise (CCW) toward the barrier, and continued to yaw. When the airborne vehicle was engaged with the cable barrier, the vehicle’s front end dropped. This drop caused the cables to engage the rear end of the vehicle at a position lower than its center of gravity, thus producing a “tripping” effect. The combination of pitch and roll later allowed the vehicle to roll over the cables. After barrier override, the vehicle came to rest on its right side at the bottom of the fill slope and laterally back from post no. 21, or approximately 41.3 m (135 ft - 4 in.) downstream from the impact point. Barrier damage consisted mostly of deformed line posts, stretched cable, and soil failure near the slope break point, as shown in Figure 8. The maximum permanent set deflection of the upstream anchor was 45 mm (1.75 in.). The entire vehicle exterior and occupant compartment was severely damaged, as expected from a rollover on a steep slope, as shown in Figure 9.

Figure 8. Barrier Damage – Test No. CS-1
The forces transmitted to the upstream, driven, steel-post anchorage and the upstream anchor displacements were measured. Load cells were installed within each cable at both ends to monitor axial forces transferred to the anchors. The results of the load cell data is summarized in Table 1. As expected, the upstream anchor produced the higher load pattern. The maximum forces acting on the upstream and downstream anchors were 108.4 kN (24.36 kips) and 90.7 kN (20.39 kips), respectively. A string potentiometer was also installed at the upstream end to record dynamic displacement of the steel-post anchorage, which recorded a maximum dynamic displacement of 66 mm (2.6 in.). Unfortunately, vehicle rollover and barrier override did not allow for maximum tensile forces to be developed within the cable barrier system.

### Table 1. Load Cell Results – Test No. CS-1

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Location</th>
<th>Maximum Cable Load</th>
<th>Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Combined Cable Load</td>
<td>Upstream End</td>
<td>108.4 kN 24.36 kips</td>
<td>0.303</td>
</tr>
<tr>
<td></td>
<td>Downstream End</td>
<td>90.7 kN 20.39 kips</td>
<td>0.303</td>
</tr>
<tr>
<td>Maximum Load in Top Cable</td>
<td>Upstream End</td>
<td>38.9 kN 8.74 kips</td>
<td>0.303</td>
</tr>
<tr>
<td></td>
<td>Downstream End</td>
<td>29.2 kN 6.57 kips</td>
<td>0.302</td>
</tr>
<tr>
<td>Maximum Load in Middle Cable</td>
<td>Upstream End</td>
<td>30.8 kN 6.93 kips</td>
<td>0.302</td>
</tr>
<tr>
<td></td>
<td>Downstream End</td>
<td>38.2 kN 8.58 kips</td>
<td>0.284</td>
</tr>
<tr>
<td>Maximum Load in Bottom Cable</td>
<td>Upstream End</td>
<td>49.3 kN 11.09 kips</td>
<td>0.288</td>
</tr>
<tr>
<td></td>
<td>Downstream End</td>
<td>34.2 kN 7.68 kips</td>
<td>0.303</td>
</tr>
</tbody>
</table>

During the test, the three-cable barrier with steel posts spaced on 4.88 m (16 ft) centers and installed approximately 305 mm (12 in.) away from a 1.5H:1V fill slope was unable to safely contain and smoothly redirect the vehicle. The vehicle penetrated the barrier system and did not remain upright after collision with the guardrail. Once the impacting vehicle became completely airborne, vehicle stability became a concern. Furthermore, lateral cable forces likely generated a destabilizing moment against the vehicle, thus contributing to vehicle roll over the cables. No detached elements or fragments showed potential for penetrating the occupant compartment nor presented undue hazards to other traffic. However, test no. CS-1 was deemed unacceptable according to the TL-3 safety performance criteria found in NCHRP Report No. 350.

### 8 BARRIER VII COMPUTER SIMULATION VISITED

The safety performance of a standard three-cable barrier system installed adjacent to a 1½H:1V fill slope was examined through a full-scale crash test with a 2,000-kg (4,409-lb) pickup truck. Field performance of the three-cable barrier system was not accurately predicted by initial BARRIER VII modeling. The results from test no. CS-1 revealed that the vehicle did not significantly travel down the slope but rather remained relatively parallel to the level terrain, which showed poor correlation between simulation and field testing. Thus, an investigation and review was made using BARRIER VII simulations for level terrain conditions and test no. CS-1. It was concluded that the resultant lateral forces in the cables had nearly achieved their peak values. Therefore, a correlation between the maximum lateral barrier displacement and the maximum longitudinal anchor displacement could be compared to the simulation results from the
level terrain BARRIER VII modeling effort. This comparison suggested that the crash-tested cable barrier system was stiffer than observed in the computer simulations. The source of increased barrier stiffness was attributed to an increased stiffness of the steel-post anchorage system. Thus, the initial level terrain BARRIER VII models were modified to account for an increased anchor stiffness of 1.5 times that used in the original simulation models. With this revision, the barrier model was deemed capable of predicting barrier performance over steep fill slopes.

With the poor barrier performance, design modifications were deemed necessary. The three-cable barrier system could be significantly improved by reducing lateral barrier deflection and reducing the potential for vehicle travel onto the embankment. Thus, barrier stiffness and strength were increased by decreasing post spacing from full-post spacing to quarter-post spacing, or 4.88 m (16 ft) to 1.22 m (4 ft). From another BARRIER VII simulation effort, a maximum lateral barrier deflection of 1,588 mm (62.5 in.) was predicted, which corresponded to reduced vehicle c.g. drop of approximately 254 mm (10 in.). When evaluating the upper limit for the lateral barrier deflection, a 2,179-mm (85.8-in.) lateral displacement yielded a potential vehicle c.g. drop of nearly 635 mm (25 in.). In order to further reduce potential vehicle c.g. drop, an increased barrier offset away from the slope break point was evaluated. With the barrier system placed 1.22 m (4 ft) away from the slope break point, the vehicle penetration onto the slope was approximately 0.9 m (3 ft). A reduction in vehicle encroachment onto the slope results in reduced vehicle c.g. drop and decreased risk for vehicle rollover and barrier override. For a 0.9-m (3-ft) lateral penetration onto the slope, the vehicle c.g. drop is essentially limited to 0.3 m (1 ft). Thus, the lateral barrier offset away from the slope break point was also increased from 0.3 m (1 ft) to 1.22 m (4 ft).

9 MODIFIED BARRIER SYSTEM (1.22-m Lateral Offset – 1.22-m Post Spacing)

The modified cable barrier system was 150.57 m (494 ft) long and largely consisted of wire rope cables, spring compensator end assemblies, threaded rod connectors, end couplers, end terminal hardware, and foundation anchorages. These components are mostly depicted in Figures 10 through 12, while additional design details are provided in Reference (10).

Three 19-mm (¾-in.) diameter, 3x7 wire ropes were used for the rail elements. The cables were positioned with centerline heights of 762 mm (30 in.), 686 mm (24 in.), and 610 mm (24 in.), which were supported by ninety-two posts spaced on 1.22 m (4 ft) centers. The cables were attached to line posts using three 8-mm (5/16-in.) diameter, standard cable hook bolts. The line posts were 1,600-mm (63-in.) long and fabricated from ASTM A36 S76x8.5 (S3x5.7) steel sections. The post sections incorporated a 203-mm x 610-mm x 6-mm (8-in. x 24-in. x ¼-in.) soil bearing plate that was welded to the backside flange and were embedded 762 mm (30 in.) into the soil. The guardrail posts were installed approximately 1.22 m (4 ft) away from the slope break point of a 1½H:1V fill slope.

The cables were tensioned using spring compensators, and the ends of the cables were fitted with threaded rods that terminated at the foundation anchorage systems. The threaded rods were attached to the end anchorage with three 51-mm (2-in.) diameter washers and two 19-mm (¾-in.) diameter Grade 5 nuts. As noted previously, traditional reinforced concrete anchorages have also been used to support the first slipbase post on each end. Thus, an alternative anchorage was used for the first slipbase post on each end. Details for the modified foundation for the first slipbase post is contained in Reference (10).

One steel-post anchorage was used on each end (post nos. 1 and 92) and fabricated from an ASTM A36 W152x37.2 (W6x25) steel section and embedded in the soil to a depth of 2,438 mm (96 in.). A 13-mm (½-in.) thick by 610-mm (24-in.) square, ASTM A36 steel soil plate was welded to the post flange by a series of 10-mm (⅜-in.) fillet welds. Furthermore, a steel cable anchor bracket was bolted to a 13-mm x 368-mm x 229-mm (½-in. x 14½-in. x 9-in.) steel plate that was welded to the top of the vertical anchor post. A steel cable anchor bracket was bolted to the top mounting plate using with four 19-mm (¾-in.) diameter x 64-mm long (2.5-in.) Grade 5 hex head bolts.

Terminal post nos. 2 and 91 were configured with ASTM A36 S76x8.5 (S3x5.7) steel sections measuring 838 mm (33 in.) long for the upper slipbase region and ASTM A36 W152x13.4 (W6x9) steel sections measuring 1,829 mm (72 in.) long for the lower foundation region. The foundation post was embedded to a depth 1,778 mm (70 in.). A slipbase plate was welded to the bottom of the upper slipbase section and the top of the lower foundation section. Four 13-mm diameter (½-in.) x 51-mm (2-in.) long ASTM A307 bolts with nuts and washers attached the upper and lower segments together. Terminal post nos. 3 through 7 and 86 through 90 were also slipbase posts that were configured with ASTM A36 S76x8.5 (S3x5.7) steel sections and ASTM A36 W152x13.4 (W6x9) steel sections. These posts were identical to post nos. 2 and 91, except that the special cable bracket was replaced with three cable hooks.

A 48.7-m (160-ft) long x 6.1-m (20-ft) wide pit was excavated behind the cable barrier system. The front profile of the sloped section was 6.1 m (20 ft) wide by 4.1 m (13.33 ft) deep, thus creating a 1½H:1V fill slope.
10 TEST NO. CS-2 (2,032 kg – 99.1 km/h – 23.6 deg)

The 2,032-kg (4,481-lb) pickup truck impacted the three-cable guardrail system at a speed of 99.1 km/h (61.6 mph) and at an angle of 23.6 degrees. The target impact point was to occur between post nos. 31 and 32, or 305 mm (1 ft)
upstream from the centerline of post no. 32, as shown in Figure 13. Actual vehicle impact occurred at post no. 32. Early in the impact event, the three cables were wrapped around the left-front corner of the pickup truck. After the left-rear tire became airborne as it traversed completely over the slope breakpoint, the pickup truck encountered significant roll toward the barrier. Later, the pickup truck reached a maximum negative roll of -24.5 degrees. At 0.750 sec, the pickup truck became parallel to the barrier system with a resultant velocity of 62.7 km/h (39.0 mph). The pickup truck exited the system at a trajectory angle of 13.2 degrees and at a resultant velocity of 54.2 km/h (33.7 mph). The pickup truck came to rest 42.9 m (140.9 ft) downstream from the impact point.

![Figure 13. Test Vehicle and Target Impact Location – Test No. CS-2](image)

Barrier damage was moderate and consisted mainly of deformed line posts, stretched cable, and soil failure, as depicted in Figure 14. The length of vehicle contact was approximately 19.7 m (64 ft - 8 in.), which spanned from the upstream edge of post no. 32 through the upstream edge of post no. 48. The maximum permanent set deflection of upstream and downstream anchors was approximately 51 mm (2 in.) and 38 mm (1.5 in.), respectively. All three cables were detached from post nos. 33 through 48. The upstream and downstream steel-post anchorages encountered slight permanent set deformations. The maximum lateral permanent set post deflection was 578 mm (22.75 in.) at the centerline of post no. 36. The maximum lateral dynamic cable deflection was 3,163 mm (124.5 in.), as determined from high-speed digital video analysis. The working width of the system was found to be 3,318 mm (130.6 in.).

![Figure 14. Barrier Damage – Test No. CS-2](image)

Exterior vehicle damage was moderate and concentrated on the left-front corner and left side of the vehicle, as shown in Figure 15. The left-front quarter panel buckled inward. The left-front corner of the bumper was bent toward the engine compartment. The left-front tire was punctured, and the right-front wheel well was severely damaged. The front of the engine hood was slightly deformed upward. Significant damage due to cable friction was found on the entire left side of the vehicle. Occupant compartment deformations to the left side of the floorboard were judged insufficient to cause serious injury to the vehicle occupants. Maximum longitudinal deflections of 6.35 mm (0.25 in.) were located along the front of the driver’s side floor panel. A maximum lateral deflection of 13 mm (0.5 in.) was located on the right-front
corner of the driver’s side floor panel. Maximum vertical deflections of 6.35 mm (0.25 in.) were located along the right side of the driver’s side floor panel near the front of the vehicle and in the center of the driver’s side floor panel as well.

![Image of vehicle damage](image)

**Figure 15. Vehicle Damage – Test No. CS-2**

The longitudinal and lateral occupant impact velocities were -4.16 m/s (-13.64 ft/s) and 3.42 m/s (11.22 ft/s), respectively. The maximum 0.010-sec average occupant ridedown deceleration in the longitudinal and lateral directions were -5.73 g’s and 6.96 g’s, respectively. Both the occupant impact velocities (OIVs) and occupant ridedown decelerations (ORDs) were within the suggested limits provided in NCHRP Report No. 350. The THIV and PHD values were determined to be 5.14 m/s (16.86 ft/s) and 8.27 g’s, respectively.

Once again, the forces transmitted to the upstream, driven, steel-post anchorage and the upstream anchor displacements were measured. Load cells were only installed within each cable on the upstream to monitor axial forces transferred to the anchor. The results of the load cell data is summarized in Table 2. The maximum forces acting on the upstream anchor were 109.60 kN (24.64 kips). A string potentiometer was also installed at the upstream end to record dynamic displacement of the steel-post anchorage. As measured with a string potentiometer, the driven steel-post anchorage measured a maximum dynamic displacement of 77 mm (3.0 in.).

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Maximum Cable Load</th>
<th>Time sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Combined Cable Load</td>
<td>109.60</td>
<td>0.37</td>
</tr>
<tr>
<td>Maximum Load in Top Cable</td>
<td>34.56</td>
<td>0.37</td>
</tr>
<tr>
<td>Maximum Load in Middle Cable</td>
<td>36.79</td>
<td>0.37</td>
</tr>
<tr>
<td>Maximum Load in Bottom Cable</td>
<td>39.41</td>
<td>0.37</td>
</tr>
</tbody>
</table>

During the test, the three-cable barrier with steel post spaced on 1.22-m (4-ft) centers, or quarter-post spacing, and installed approximately 1.22 m (4 ft) away from a 1½H:1V fill slope was able to safely contain and redirect the vehicle with controlled lateral displacements of the barrier system. No detached elements or fragments showed potential for penetrating the occupant compartment nor presented undue hazard to other traffic. Deformations of, or intrusion into, the occupant compartment that could have caused serious injury did not occur. The vehicle did not penetrate nor ride over the barrier and remained upright during and after the impact event. The longitudinal occupant impact velocity and ridedown deceleration were within the recommended limits. Therefore, test no. CS-2 was deemed acceptable according to the TL-3 safety performance criteria found in NCHRP Report No. 350.

**11 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS**

A 147.82-m (485-ft) long, standard three-cable guardrail system was constructed with the line posts positioned 0.3 m (1 ft) away from a 1½H:1V fill slope. One 2000P full-scale crash test was conducted and found to be unacceptable according to the TL-3 safety performance criteria presented in NCHRP Report No. 350. Lateral vehicle penetration over the slope allowed the vehicle to become completely airborne, which resulted in the front-impact side of the vehicle dropping below the vehicle e.g. Thus, the barrier’s reductive force was applied below the e.g. of the vehicle, where the roll moment induced a “tripping” effect on the vehicle. The destabilizing moment caused the vehicle to roll over the
The poor performance observed in test no. CS-1 warranted design modifications. Consequently, computer simulation with BARRIER VII was used to investigate barrier performance and evaluate several design modifications. This analysis demonstrated that a post-spacing reduction from 4.88 m (16 ft) to 1.22 m (4 ft) provided increased lateral barrier stiffness and strength as well as resulted in decreased lateral barrier deflections. The post-spacing reduction was coupled with an increased lateral barrier offset away from the slope break point. An increased lateral offset from 0.3 m (1 ft) to 1.22 m (4 ft) would limit vehicle penetration onto the slope, thus reducing the potential vehicle c.g. drop. These design changes were believed to significantly improve the safety performance of standard three-cable guardrail systems installed adjacent to a 1½H:1V fill slopes. Thus, full-scale crash testing was recommended for the modified cable barrier to verify its safety performance.

Another research objective involved the redesign of common cable end terminal anchorages. Three alternative anchorages were investigated and included: (1) a reduced-size, reinforced concrete block; (2) a reinforced concrete foundation placed within a drilled shaft; and (3) a driven steel-post anchorage. The dynamic bogie testing results and finite element modeling indicated that the driven steel-post anchorage was the weakest configuration and provided axial resistance and displacements slightly outside of the targeted ranges. However, the driven steel-post anchorage was still deemed viable and able to provide the necessary cable resistance. During test no. CS-1, the steel-post anchorage performed as intended and resisted significant cable tension. The satisfactory performance of the steel-post anchorage showed that the reduced-size concrete block and drilled concrete shaft would also withstand the tensile loads imparted to the anchorages due to their increased capacity. However, it should be noted that the impacting vehicle was not contained nor smoothly redirected. Thus, the three alternative anchorages would continue to be evaluated in subsequent crash testing at interior regions of the barrier system as well as near the terminal end.

Following the failed pickup truck crash test and using the 2-D BARRIER VII computer simulation program, the standard three-cable barrier system was analyzed and modified in order to improve barrier performance and eliminate concerns for override and vehicle rollover. System modifications included a reduction in post spacing from 4.88 m (16 ft) to 1.22 m (4 ft) or from standard post spacing to quarter-post spacing. In addition, the lateral barrier offset away from the slope break point was increased from 0.3 m (1 ft) to 1.22 m (4 ft). The modified three-cable barrier system was constructed near the 1½H:1V fill slope and subjected to full-scale crash testing. A second 2000P crash test was successfully performed on the modified cable barrier system according to the TL-3 safety performance criteria presented in NCHRP Report No. 350. Additional details, test results, evaluation findings, and discussion regarding test no. CS-2 are provided in Reference (10). The test results indicated that this modified barrier design should be suitable for use on Federal-aid highways when installed 1.22 m (4 ft) away from the slope break point of a 1½H:1V fill slope.

It should be noted that alternative designs may allow for a closer barrier positioning relative to the slope break point. However, any significant changes to the modified cable barrier system when placed adjacent to steep slopes would require future research, including additional analysis, design, computer simulation, and full-scale vehicle crash testing.

12 ACKNOWLEDGEMENTS

The authors wish to acknowledge several sources that contributed to this project: (1) Midwest States Pooled Fund Program for sponsoring this project; (2) Florida Wire and Cable as well as Bekaert Corporation for donating cable materials; and (3) MwRSF personnel for constructing barrier components and systems as well as conducting bogie tests and crash tests.

13 DISCLAIMER

The contents of this paper reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views nor policies of the State Highway Departments participating in the Midwest States Pooled Fund Research Program nor the Federal Highway Administration. This paper does not constitute a standard, specification, or regulation.

14 REFERENCES


Abstract: The Arizona Department of Transportation has successfully used rubberized asphalt binders, specified and referred to as asphalt rubber, for several decades to reduce cracking. Over a period of several decades from the 1970’s to the 1990’s many research studies and papers documented the aging characteristics of asphalt rubber and neat asphalt (bitumen) in terms of penetration and viscosity testing. Data and findings from these studies are used to develop asphalt rubber and neat asphalt aging properties. The aging properties are described in terms of routine penetration and viscosity type asphalt aging comparisons. In addition PG grading G* and phase angle data are also represented over a four year period in the late 1990’s. The PG grading G* and phase angle are examined and related in a novel way to the routine asphalt test. The objective of this paper is to use this rather large body of accumulated asphalt test data to describe the rate of asphalt rubber and neat asphalt aging. The rate of asphalt aging in Arizona is related to the degree of pavement cracking observed in the field over the years.

**KEYWORDS:** Rubberized asphalt, rubber, asphalt, aging, cracking
Pavement Aging Properties of Rubberized Asphalt and Neat Asphalt

George B. Way

Dr. Krishna Prapoorna Biligiri

1Rubberized Asphalt Foundation, Scottsdale, Arizona, USA

Email for correspondence: wayouta@cox.org

2Indian Institute of Science, Kharagpur, India kpb@civil.iitkgp.ernet.in

1 INTRODUCTION

The Arizona Department of Transportation (ADOT) conducted numerous research and special studies of rubberized asphalt, specified and referred to as asphalt rubber; and neat asphalt (bitumen) a product of the petroleum refining industry. These various research and special studies were conducted and reported on over a 30 year period (Way 1972), (Way 1975), (Way 1976) (Way 1978), (Way 1979), (Way 1980), (Way1997), (Way 1999), (Way et al. 2012a), (Forstie & Way 1977), (Stroud 1991), (Bouldin et al. 1994), (Arizona 1988), (Lytton 1993), (Reese 1998) and (Sousa et al. 1995). Although these various studies were directed at a myriad of topics many of them contained original asphalt rubber binder and neat asphalt properties and later aged properties derived from asphalt extracted from cores. The studies included pavements that ranged in age from new construction asphalt properties to extracting asphalt from 20 year old pavements. The intent of this paper is to show how the testing properties of the asphalt rubber and neat asphalt changed with aging in the field. The asphalt tests include the traditional tests; the penetration at 25°C, the micro-viscosity at 25°C, absolute viscosity at 60°C, kinematic viscosity at 135°C. The penetration, absolute viscosity and kinematic viscosity tests were specified and tested in accordance with American Association of State Highway and Transportation Officials (AASHTO) requirements which are similar to ASTM International testing requirements. Besides traditional testing the binders, other testing to derive the G* and phase angle were conducted in accordance with the PG grading protocol (AASHTO 2005a). Results of the core binder G* and phase angle tests were compared to a data base of the G* and phase angle for new asphalt tested in the late 1990’s. This G* and phase angle data base contains the original unaged, Rolling Thin Film Oven (RTFO) and Pressure Aging Vessel (PAV), G* and phase angle for hundreds of asphalts.

2 ARIZONA HISTORY OF ASPHALT AND ASPHALT RUBBER

Arizona used a variety of neat asphalts from 1966 to 2000. Over this long period of time the asphalt grades that were used or tried included 200/300, 120/150 or 85/100, 60/70 and 40/50 penetration [17]; AR2000 and AR4000; AC 30 and AC 40 (AASHTO 2005a); PBA 3, PBA 4, PBA 6, PBA 7 (PCCAS 1991), (Caltrans 2006) and since 1997 the PG grades such as PG 70-10 (AASHTO 2005b). These various grades of asphalt were used in dense graded mixes that were either designed using the Hveem or Marshall of mix design. Typically the average asphalt binder content 5.4 percent by weight of the mix. The design air void content was 5 percent and the average VMA was 17 percent. Neat asphalt was also in open graded mixes. The open graded mix has an average binder content of 6.0 percent by weight of the mix. The air void content typically was 20.8 percent and the average VMA was 29.1 percent.

Asphalt rubber binder is a mixture of 80 percent hot paving grade asphalt (bitumen) with 20 percent ground tire rubber produced from waste tires. Asphalt rubber binder is used in either a gap graded mix or open graded mix (Sousa, 2006). The asphalt rubber gap graded mix has an average binder content of 7.3 percent by weight of the mix. The design air void content was 5 percent and the average VMA was 20 percent. The asphalt rubber open graded mix has an average binder content of 9.2 percent by weight of the mix. The air void content typically was 20.2 percent and the average VMA was 32.5 percent. Figure 1 is a comparison of the average Hveem and Marshall dense graded mixes gradation to the average asphalt rubber gap graded mixes gradation. Figure 2 is a comparison of the gradation of the open graded mixes with neat asphalt to the asphalt rubber open graded mixes (Way 2012b).
3 DESCRIPTION OF ASPHALT TESTS

The asphalt tests in the data base include the penetration 25°C (AASHTO 2005c), the micro-viscosity at 25°C (ASTM 1961), (Fink & Griffen 1961), absolute viscosity at 60°C (AASHTO 2005d), kinematic viscosity at 135°C (AASHTO 2005e). The penetration, absolute viscosity and kinematic viscosity tests were specified and tested in accordance with AASHTO requirements which are similar to ASTM testing requirements. The micro-viscometer tests were conducted in a manner consistent with published research literature and equipment manufacturer suggested test procedures. Micro-viscosities of unaged and aged asphalt were measured on a Hallikainen sliding plate micro-viscometer at a constant temperature of 25°C. Glass plates were used with unaged asphalt except for those viscosities above 5 MPa.s. For unaged asphalts with viscosities above 5 MPa.s and for all aged asphalts from cores steel plates were used. For all asphalt samples that were tested successively lighter weights, which imposed smaller shear rates. Usually four shear rates were imposed on a sample. Based on the shear rates and shear stresses, the micro-viscosity was determined for a shear rate of 0.05 sec⁻¹, (Peters 1975).

Asphalt tests were conducted on the asphalt binder sample from the tank at the time of construction, referred to as the original unaged asphalt. None of the asphalts in the data base were specified to be modified and virtually all are unmodified, neat asphalts, albeit some modification may have been done but not reported. Later as required by the various research studies cores of the pavement were taken and the asphalt extracted by means of an ADOT extraction test ( Soxhlet 1996). In all 157 different test sites were cored and asphalt recovered. The tested asphalt varied in age from 1 month to 264 months in age.
4 PENETRATION AND VISCOSITY DATA ANALYSIS

The penetration and viscosity data were analysed to estimate the rate of aging. Equations for the aging of the neat asphalt were derived for penetration and viscosity. Table 1 shows the derived equations. Table 2 converts the values from the equations to index values at one, five, ten and twenty years. By multiplying the tabled index values by the original penetration and viscosity values can be estimated over a period of years. Table 2 demonstrates the viscosity at 25°C increases more rapidly than the 60°C, and the 60°C viscosity increases more rapidly than the 135°C.

Table 1. Equations derived from asphalt recovered from field

<table>
<thead>
<tr>
<th>Asphalt Test</th>
<th>Original Value Unaged</th>
<th>Equation – Age in Months</th>
<th>Number of Observations</th>
<th>Correlation R²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Penetration 25°C</td>
<td>100</td>
<td>-10.6*LN(Age)+62.42</td>
<td>157</td>
<td>0.403</td>
</tr>
<tr>
<td>Micro-Viscosity 25C, 10⁷ Pa·s</td>
<td>0.150</td>
<td>0.50047<em>e⁻⁰⁶⁰⁰⁴³</em>Age</td>
<td>157</td>
<td>0.559</td>
</tr>
<tr>
<td>Absolute Viscosity 60C, Pa·s</td>
<td>170.0</td>
<td>619.6<em>e⁻⁰⁶⁰⁰⁴³</em>Age</td>
<td>157</td>
<td>0.438</td>
</tr>
<tr>
<td>Kinematic Viscosity 135C, Pa·s</td>
<td>0.310</td>
<td>45.44<em>e⁻⁰⁶⁰⁰⁴³</em>Age</td>
<td>157</td>
<td>0.408</td>
</tr>
</tbody>
</table>

Table 2. Index values derived from equations in Table 1

<table>
<thead>
<tr>
<th>Age in Months</th>
<th>Penetration Index 25°C</th>
<th>Micro-Viscosity Index 25°C</th>
<th>Absolute Viscosity Index 60°C</th>
<th>Kinematic Viscosity Index 135°C</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>0.62</td>
<td>4.3</td>
<td>4.3</td>
<td>1.6</td>
</tr>
<tr>
<td>60</td>
<td>0.45</td>
<td>10.7</td>
<td>8.4</td>
<td>1.9</td>
</tr>
<tr>
<td>120</td>
<td>0.37</td>
<td>33.5</td>
<td>19.8</td>
<td>2.6</td>
</tr>
<tr>
<td>180</td>
<td>0.33</td>
<td>104.8</td>
<td>46.4</td>
<td>3.5</td>
</tr>
<tr>
<td>240</td>
<td>0.29</td>
<td>327.6</td>
<td>108.7</td>
<td>4.5</td>
</tr>
</tbody>
</table>

5 DATA ANALYSIS NEAT ASPHALT

Besides reviewing the penetration and viscosity versus time in months, i.e., aging another portion of this study is to attempt to relate the Penetration to the micro-viscosity at 25°C. Then relate the micro-viscosity to the PG grading PAV Phase angle since the phase angle represents the viscous portion of the mechanical analysis. If all of these relationships could be reasonably derived then the following scheme could be used to estimate the PG grading PAV G* and Phase angle from the 25 C penetration and related micro-viscosity as diagrammatically shown as follows:

Penetration → Microviscosity → Phase Angle (θ) → G*

As a first step the 25°C penetration of the asphalt was correlated to the 25°C micro-viscosity. The correlation equation (1) is shown as follows and Table 3 shows the micro-viscosity predicted from the penetration:

\[ \text{Micro-viscosity (MPa·s)} = 354.4\times(\text{Penetration})^{1.848} \]  (1)

\[ \text{R}^2 = 0.8121 \quad \text{N} = 157 \]

In the next step the micro-viscosity at 25°C was related to phase angle at 25°C from tests conducted on the neat asphalt recovered from the cores and additional unaged asphalts. The resultant equation (2) was found and predicted values are shown in Table 3:

\[ \text{Phase angle (θ)} = -6.059\times\text{Ln(micro-viscosity)}+61.25 \]  (2)

\[ \text{R}^2 = 0.8530 \quad \text{N} = 157 \]
Table 3. Micro-viscosity predicted from penetration; Phase angle predicted from Micro-viscosity; G* predicted from Phase Angle

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>354</td>
<td>12</td>
<td>1200000</td>
<td>60</td>
<td>0.18</td>
<td>58</td>
<td>290</td>
</tr>
<tr>
<td>2</td>
<td>99</td>
<td>19</td>
<td>340000</td>
<td>80</td>
<td>0.11</td>
<td>60</td>
<td>200</td>
</tr>
<tr>
<td>5</td>
<td>18</td>
<td>30</td>
<td>46000</td>
<td>100</td>
<td>0.07</td>
<td>63</td>
<td>116</td>
</tr>
<tr>
<td>10</td>
<td>5.0</td>
<td>37</td>
<td>13000</td>
<td>120</td>
<td>0.05</td>
<td>65</td>
<td>81</td>
</tr>
<tr>
<td>20</td>
<td>1.4</td>
<td>45</td>
<td>3000</td>
<td>150</td>
<td>0.03</td>
<td>69</td>
<td>39</td>
</tr>
<tr>
<td>40</td>
<td>0.39</td>
<td>53</td>
<td>710</td>
<td>200</td>
<td>0.02</td>
<td>71</td>
<td>27</td>
</tr>
<tr>
<td>50</td>
<td>0.26</td>
<td>55</td>
<td>490</td>
<td>250</td>
<td>0.013</td>
<td>74</td>
<td>16</td>
</tr>
</tbody>
</table>

Next step was to predict the G* from the phase angle using the original, rolling thin film oven (RTFO) and pressure aging vessel (PAV) values obtained by routine testing in the late 1990’s and more recently 2010 (Western 2010). The following equation (3) was derived from the correlation, representative of a Blacks diagram predicted values are shown in Table 3:

\[
\text{Phase angle (}\delta\text{)} = -5.5172\text{Ln}(G^*) + 89.2146
\]

\[
R^2 = 0.9822 \quad N = 957
\]

Equation (3) is re-written to directly solve for G* as shown in Equation (4) as follows:

\[
G^* = e^{\left[\delta - 89.2146 \over -5.5172\right]}
\]

With the prediction of G* from the phase angle it was possible to calculate a predicted G*Sinδ and match these values to the percent observed cracking in the field. The ADOT estimates the percent cracking visually as part of an annual pavement management review of the Arizona state highways using a cracking picture guide, Figure 3.

Figure 3. ADOT Standard Percent Cracking Photos, Percent by Area of High Traffic Lane Cracked (Way 1979).
Reviewing the estimated G*SinΩ in terms of the degree of cracking provided the following equation (5):

\[ \text{Percent Cracking} = -14.078 + 2.492 \times \ln(G*\text{SinΩ}) \]  
\[ R^2 = 0.3904 \quad N = 157 \]  

In addition an equation (6) was developed that related G*SinΩ to the number of months of aging of the asphalt in the field.

\[ \ln(G*\text{SinΩ}) = 6.822 + 0.0192 \times (\text{Age in Months}) \]  
\[ R^2 = 0.6774 \quad N = 157 \]  

It is interesting that the G*SinΩ is better related to the age of the asphalt than to the percent cracking, albeit the level of percent cracking is very subjective in nature. In addition to predictive equations another way of representing the predicted G*SinΩ is to review how the actual percent cracking and aging compared to G*SinΩ. Table 4 shows the G*SinΩ cracking values below and above the 5000 kPa level and G*SinΩ and average percent cracking at the end of five year periods. The 5000 kPa G*SinΩ value obtained after RTFO and PAV aging. The pressure aging vessel (PAV) was adopted by Superpave to simulate the effects of long-term asphalt binder aging that occurs as a result of 5 to 10 years HMA pavement service (Strategic 1998), (Anderson et al. 1991) and (Bahia & Anderson 1995). On average from the Arizona data set the five to ten years of aging seems reasonable. Likewise the 5000 kPa G*SinΩ value also is represented to equate to about ten percent cracking. The Arizona data, albeit limited, would indicate that the 5000 kPa value does not quite represent the ten percent level but is close and could easily represent a range of five to ten percent cracking. Although predicting cracking from G*SinΩ is somewhat limited in certainty, it is clear, however that as the G*SinΩ value increases the percent cracking increases.

Table 4. G*SinΩ values in terms of predicting cracking and asphalt aging

<table>
<thead>
<tr>
<th>G*SinΩ&lt;5000 kPa</th>
<th>Average percent cracking</th>
<th>Average age of pavement months</th>
<th>Age in Months</th>
<th>Average percent cracking</th>
<th>Average G*SinΩ kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>28</td>
<td>0-60</td>
<td>0.5</td>
<td>3150</td>
<td></td>
</tr>
<tr>
<td>G*SinΩ&gt;5000 kPa</td>
<td>10.3</td>
<td>111</td>
<td>61-120</td>
<td>7.7</td>
<td>31800</td>
</tr>
<tr>
<td></td>
<td></td>
<td>121-180</td>
<td>16.7</td>
<td>51100</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>180-264</td>
<td>22.2</td>
<td>178000</td>
<td></td>
</tr>
</tbody>
</table>

6 DATA ANALYSIS ASPHALT RUBBER

A very limited special study was conducted to compare the aging of pavements constructed with asphalt rubber gap graded and open graded mixes. It is theorized that since these asphalt rubber mixes are placed with a great amount of asphalt and a thicker film thickness the asphalt binder will age slower and there will be less cracking (Way et al. 2012b). Figure 4 shows the results from cores taken from pavements with asphalt rubber and neat asphalt in the same age bracket of over 100 months in age show the asphalt rubber asphalt binder has a higher phase angle indicative of a greater portion of viscous material. Likewise the higher the phase angle the smaller the G* elastic component, in essence the asphalt rubber is a less brittle material at the same age as the neat asphalt.
Laboratory Correlation:
Phase Angle = 86.365e^-0.0042Age
\( R^2 = 0.8605 \)
n = 957

![Correlation Graph](image)

Figure 4. Correlation of Laboratory Phase Angle – Age Relationship with Field Core-Aged Binder Test Results.

Historically it has been shown that less cracking occurs over time with asphalt rubber binder than neat asphalt (Sousa et al. 2006). To further the evidence that asphalt rubber mixes are less brittle and therefore less prone to cracking over time, Figure 5 shows the percent cracking versus months of field aging for pavements with neat asphalt and asphalt rubber. As can be seen the asphalt rubber has less cracking over the 264 months of service.

![Cracking Graph](image)

Figure 5. Cracking of hot mix asphalt dense grade pavements and pavements with asphalt rubber.

7 CONCLUSIONS

The purpose of this study was to add to the body of knowledge on the aging characteristics of neat asphalt binders and asphalt rubber using more than 20 years of binder test data.

Assemble unaged and aged asphalt rubber and neat asphalt test data that constitutes a wide range of penetration, viscosity, and PG-grades, original, RTFO, and PAV aging conditions; over 157 test sections and 318 variety of asphalt cements, and over 957 test data points.

A large amount of unaged and aged neat asphalt data and a smaller amount of asphalt rubber data has been assembled and reviewed to observe the amount of asphalt aging that takes place in the field over a 20 year period. This
data base is of benefit to show the manner and degree to which asphalt aging and corresponding penetration and viscosity of the neat asphalt changed over time the degree of cracking increases.

A scheme was developed to estimate Phase Angle and G* from penetration and micro-viscosity data (Penetration → Microviscosity → Phase Angle → G*). From this analysis scheme reasonable equations and correlations were derived to describe the rate of aging for neat asphalt in terms of G* and phase angle.

Relationships of penetration and viscosity of PG grades at 25°C were developed for the phase angle as this represents the viscous component of binder characteristics.

The 5000 kPa G*Sinâ value obtained after RTFO and PAV aging is represented to equate to the degree of aging after five to ten years in the field. On average from the Arizona data set the five to ten years of aging seems reasonable.

The Arizona data, albeit limited, would indicate that the 5000 kPa value does not quite represent the ten percent level of cracking but is close and could easily represent a range of five to ten percent cracking. Although predicting cracking from G*Sinâ is somewhat limited in certainty, it is clear, however that as the G*Sinâ value increases the percent cracking increases.

Asphalt rubber asphalt binder has a higher phase angle indicative of a greater portion of viscous material. Likewise the higher the phase angle the smaller the G* elastic component, in essence the asphalt rubber is a less brittle material at the same age as the neat asphalt.

Evidence from this study demonstrated that asphalt rubber mixes are less brittle and therefore less prone to cracking over time;

8 REFERENCES


Rubberized Asphalt Open Graded Friction Course
History and Worldwide Use

Abstract: The worldwide use and technology of rubberized asphalt open graded friction course mixes has grown and expanded since it was first introduced in Arizona in the mid 1980’s to its present use in not only the United States but also Portugal, Brazil and China. Rubberized asphalt open graded friction courses (RAFC) are placed as the top course wearing surface. Their function is multifaceted and includes providing a wet weather surface with very good friction properties (skid resistance), reduce reflective and fatigue type cracking, smooth riding surface in terms of the international roughness index and a surface that dampens the tire/pavement noise, and reducing emission rates of tire wear. RAFC’s are composed of a high quality clean open grade aggregate. The binder content is typically in the range of 9 to 10 percent by weight of the aggregate and placed from 12.5 mm to 25 mm in thickness. The rubberize asphalt binder is composed of typically 80 percent asphalt (bitumen) and 20 percent recycled tire rubber. The objective of this paper is to review and summarize the use of RAFC’s in various countries and to report on the technical research findings that have buoyed the use of this unique material.

KEYWORDS: Rubberized asphalt, asphalt, recycled tire rubber, open graded
Rubberized Asphalt Open Graded Friction Course History and Worldwide Use

George B. Way\textsuperscript{1}

\textsuperscript{1}Rubberized Asphalt Foundation, Scottsdale, Arizona, USA
Email for correspondence: wayouta@cox.net

Krishna Prapoorna Biligiri, Research Scientist, Indian Institute of Science, Kharagpur, India
kpb@civil.iitkgp.ernet.in

Kamil Kaloush, Associate Professor, Arizona State University, Tempe, Arizona, USA,
kamil.kaloush@asu.edu

Jorge Sousa, CEO, Consulpav International, Walnut Creek, California, USA, jmbsousa@aol.com

Angelo Pinto, Division Head, Rio de Janeiro State Highway Department, Brazil, angpint@uol.com.br

Rongji Cao, Vice President, Jiangsu Transportation Research Institute, Nanjing, China, crj@ti.js.cn

1 INTRODUCTION

Open Graded Friction Courses (OGFC) began to be used in California and Arizona in the early 1950’s (Morris & Scott 1973). The primary reason for using this material was to provide a surface with good skid resistance, good smooth ride and appearance. The original open graded mixtures consisted mostly of a single size 2.36 mm aggregate with approximately 5.5 to 6.5 percent paving grade asphalt. The resultant hot mix was mixed in a hot plant and placed with a laydown machine at a thickness of approximately 12.5 mm. From the 1950’s to about 1985 these open graded hot mixes were placed on numerous pavements, however since they were open graded they were prone to mechanical raveling, Figure 1. In the 1970’s the Federal Highway Administration suggested that states make greater use of open graded friction course mixes (FHWA 1974). Many states experimented with these mixes and found that they did not do well in areas with snow and ice and thus the use of such mixes virtually disappeared except in California and Arizona. In the 1980’s Arizona began to use open graded friction course with a type of rubberized asphalt called asphalt rubber (AR) binder. Asphalt rubber open graded friction courses (ARFC) are placed as the top course wearing surface. Their function is multifaceted and includes providing a wet weather surface with very good friction properties (skid resistance), reduce reflective and fatigue type cracking, smooth riding surface in terms of the international roughness index and a surface that dampens the tire/pavement noise, and reducing emission rates of tire wear. ARFC’s are composed of a high quality clean open graded aggregate. The binder content is typically in the range of 9 to 10 percent by weight of the aggregate and they are typically placed from 12.5 mm to 25 mm in thickness. The asphalt rubber binder is composed of typically 80 percent asphalt (bitumen) and 20 percent recycled tire rubber. The objective of this paper is to review and summarize the use of ARFC’s in various countries and to report on the technical research findings that have buoyed the use of this unique material. In addition the report contains some information about the ARFC’s mechanical properties, noise properties and various performance measurements.

Figure 1. Raveled open graded friction course without asphalt rubber binder.
2 ASPHALTRUBBER OPEN GRADED FRICTION COURSE MIXES

Asphalt rubber began to be used as an asphalt modifier in the United States in the state of Arizona in the late 1960’s. In 1978 it was patented by two asphalt supplier companies in Arizona (Heitzman 1992). In 1994 ASTM established two standards that define and specify AR (ASTM 2011a), (ASTM 2011b). The ASTM defines AR as a mixture of at least 15% ground tire rubber derived from scrap tires and 75% hot paving grade asphalt (bitumen). The Arizona Department of Transportation (ADOT) began using AR binder as a chip seal coat binder in the 1970’s (Scofield 1989). Later in the 1980’s the City of Phoenix, Arizona began using a hot mix with AR binder. They called this hot mix an AR gap graded mix (ARAC). Also in the 1980’s the ADOT developed an open graded hot mix using AR as the binder (Way et al. 2012) and called this mix an AR open graded friction course (ARFC). Figure 2 shows the ARFC open graded gradation and binder content compared to a typical ARFC without asphalt rubber.

![Image](open_graded_frcourse.png)

Figure 2. Open graded friction course gradation and binder content with paving grade asphalt or with asphalt rubber binder.

As can be seen the ARFC has a much greater amount of binder which virtually eliminates the potential of the mix to ravel, it also reduces the incidence of reflective cracking while maintaining excellent macro-texture and skid resistance in wet weather (Zareh & Way 2009). It was also noticed that ARFC reduced the tire/pavement noise to such a degree that both City of Phoenix and ADOT conducted research studies to document the degree of tire/pavement noise reduction (Scofield 2003). This noise reduction was so significant that the ADOT covers all of its concrete pavements with ARFC to reduce noise, provide good skid resistance and a smooth ride (Gruner & Assaf 1990). A significant study conducted by both the California Department of Transportation (Caltrans) and ADOT (Donovan & Rymer 2003) compared the tire/pavement noise for various pavement surfaces and the ARFC was found to be the least noisy, Table 1.

<table>
<thead>
<tr>
<th>Location</th>
<th>Material</th>
<th>Additional Notes</th>
<th>SI Level (dBA)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AZ Marc 10</td>
<td>ARFC</td>
<td></td>
<td>96.6</td>
</tr>
<tr>
<td>AZ Marc 202</td>
<td>ARFC</td>
<td>Best Condition</td>
<td>97.4</td>
</tr>
<tr>
<td>CA SBd 40</td>
<td>RAC (Type O)</td>
<td>High Binder</td>
<td>98.4</td>
</tr>
<tr>
<td>AZ Marc 10</td>
<td>P-ACFC</td>
<td></td>
<td>98.7</td>
</tr>
<tr>
<td>CA Sol 80</td>
<td>DGAC</td>
<td></td>
<td>101.7</td>
</tr>
<tr>
<td>CA Ker 58</td>
<td>PCC</td>
<td>New Broom (Longitudinal)</td>
<td>101.8</td>
</tr>
<tr>
<td>AZ Marc 202</td>
<td>PCC</td>
<td>Longitudinal Tine</td>
<td>102.0</td>
</tr>
<tr>
<td>CA SM 84</td>
<td>Chip Seal</td>
<td></td>
<td>105.0</td>
</tr>
<tr>
<td>AZ Marc 202</td>
<td>PCC</td>
<td>New Transverse Tine</td>
<td>107.1</td>
</tr>
<tr>
<td>AZ Marc 202</td>
<td>PCC</td>
<td>New Random Transverse Tine</td>
<td>109.2</td>
</tr>
</tbody>
</table>
3 ARFC IN PORTUGAL

Portugal began to use ARFC surfaces in about 2003 to provide a smooth riding surface, good wet weather skid resistance and less tire pavement noise. Figure 3. It was reported that ARFC reduced the noise by 5-6 dBA when compared to a typical dense graded asphalt hot mix and 8-10 dBA when compared to a typical concrete surface (Ruivo 2004a) and (Ruivo 2004b).

![Figure 3. Portugal dense AC hot mix and asphalt rubber friction course surfaces.](image)

4 ARFC IN CHINA

As China’s economy has grown so has the number of scrap tires. In 2004 China produced about one hundred million scrap tires per year and by 2010 the amount had increased to 300 hundred million scrap tires per year. In the 1980’s there had been some test trials of using scrap tire in a dry form in a dense graded hot mix. Later by 2004 experiments began on using both dry process and the wet process of asphalt rubber. From 2004 to 2007 many experimental test sections and test projects were constructed using AR as a chip seal coat interlayer referred to as a stress absorbing interlayer (SAMI). Also test sections of ARAC hot mixes were constructed; both the SAMI and ARAC test sections closely approximated the materials placed in both Arizona and California. From 2007 until now the use of AR in the wet process like that used in the US has grown, Figure 4. Implementation of AR as a hot mix or seal coat has reached over 20 China provinces (Cao 2012a). Additionally specifications and guidelines for AR are becoming more common in China (Tianjin 2006), (Jiangsu 2006) and (Beijing 2006). An International conference on AR was held in Nanjing, China in 2009 (Asphalt 2009).

![Figure 4. Use of ARFC and other AR materials in China.](image)

Now that the use of AR is becoming more common in China attention has turned to measuring the tire/pavement noise of various pavement surfaces. Although the use of AR is still relatively new in China as well the measurement of tire/pavement noise is still relatively new in China; Jiangsu province in China measured the tire/pavement noise of various surfaces as previously done in Arizona and California. Figure 5 represents the first effort within China to measure the tire/pavement noise using the OBSI method (Cao 2012b). As Figure 5 shows those
pavement surfaces constructed with either ARFC or ARAC demonstrated very good tire/pavement noise reduction. It was reported that ARFC reduced the noise by 5-6 dBA when compared to a typical dense graded asphalt hot mix and 8-10 dBA when compared to a typical concrete surface.

### China OBSI Tire/Pavement Noise Measurements*

<table>
<thead>
<tr>
<th>No.</th>
<th>Road</th>
<th>Surface Type</th>
<th>Binder Type</th>
<th>Tire-Pavement Noise (dBA)</th>
<th>Construction Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Nanjing Ring G42</td>
<td>2.5cm ARFC13</td>
<td>Asphalt Rubber</td>
<td>96.0</td>
<td>2009 New Pvt</td>
</tr>
<tr>
<td>2</td>
<td>Shanghai Zhongtian</td>
<td>2.5cm OGFC13</td>
<td>High Viscosity Binder</td>
<td>98.2</td>
<td>2009 New Pvt</td>
</tr>
<tr>
<td>3</td>
<td>Ningshang G25</td>
<td>4cm AR &amp; AC13</td>
<td>Asphalt Rubber</td>
<td>98.8</td>
<td>2008</td>
</tr>
<tr>
<td>4</td>
<td>Ningshang G25</td>
<td>4cm SMA-13</td>
<td>SBS-PG70-22</td>
<td>98.9</td>
<td>2008</td>
</tr>
<tr>
<td>5</td>
<td>Haimen S336</td>
<td>4cm Smp-9.5</td>
<td>SBS-PG70-22</td>
<td>99.0</td>
<td>2005</td>
</tr>
<tr>
<td>6</td>
<td>Yuhai G15</td>
<td>4.5cm SMA-13</td>
<td>SBS-PG70-22</td>
<td>99.2</td>
<td>2005</td>
</tr>
<tr>
<td>7</td>
<td>Haimen S336</td>
<td>2.5cm OGFC-9.5</td>
<td>SBS+fiber</td>
<td>99.4</td>
<td>2005</td>
</tr>
<tr>
<td>8</td>
<td>Ningshou S55</td>
<td>2.5cm OGFC-13</td>
<td>Asphalt Rubber</td>
<td>99.5</td>
<td>2006</td>
</tr>
<tr>
<td>9</td>
<td>Nanjing Ring</td>
<td>4cm AC-13</td>
<td>Asphalt Rubber</td>
<td>99.6</td>
<td>2008</td>
</tr>
<tr>
<td>10</td>
<td>Haimen S336</td>
<td>4cm AC-13</td>
<td>PG64-22</td>
<td>99.6</td>
<td>2002</td>
</tr>
<tr>
<td>11</td>
<td>Yuhai G15</td>
<td>4cm PA-13</td>
<td>High Viscosity Binder</td>
<td>99.6</td>
<td>2005</td>
</tr>
<tr>
<td>12</td>
<td>Shanghai Zhongtian</td>
<td>4cm SMA-13</td>
<td>SBS-PG70-22</td>
<td>99.7</td>
<td>2009 New Pvt</td>
</tr>
<tr>
<td>13</td>
<td>Ningshang G25</td>
<td>4cm PA-13</td>
<td>High Viscosity Binder</td>
<td>99.9</td>
<td>2007</td>
</tr>
<tr>
<td>14</td>
<td>Shanghai Huidong Road</td>
<td>4cm AC13</td>
<td>PG64-22</td>
<td>100.1</td>
<td>2001</td>
</tr>
<tr>
<td>15</td>
<td>Haimen</td>
<td>Concrete</td>
<td>Concrete</td>
<td>103.9</td>
<td>2002</td>
</tr>
</tbody>
</table>

*Note-All measurements in 2009

**Figure 5.** China tire/pavement interface OBSI noise measurements.

### 5 BRAZIL ASPHALT RUBBER OPEN GRADED FRICTION COURSE

Brazil’s first use of an Asphalt Rubber Open Graded Friction Course was commissioned by the DER-RJ Rio de Janeiro State Highway Department on a project on highway RJ-122 in the Rio de Janeiro province. The existing pavement was paved in the seventies; its surface became extensively cracked, making the riding unsafe and uncomfortable for the users Figures 6. Furthermore, the shoulders were lacking in many areas along the highway. The traffic volume is high and mainly of trucks. For those reasons, RJ-122 was chosen for applying this new field blend asphalt rubber technique (Pinto & Sousa 2012). The overlay project consisted of widening the roadway with a dense graded hot mix leveling and reshaping layer, followed by a 4.5 cm asphalt rubber gap graded structural layer and a 2.5 cm asphalt rubber open graded surfacing, Figure 7. The completed highway RJ-122 overlay pavement has structural and functional characteristics of the highest best quality ranking in the federal and state networks. In particular the skid resistance of 0.7 which is very good. The ride smoothness also was rated as very smooth riding.

**Figure 6.** RJ 122 pavement before and after overlay placement with ARFC surface.
Figure 7. The new pavement structure, 2.5 cm of asphalt rubber open graded and 4.5 cm of asphalt rubber gap grade hot mix.

6 ASPHALT RUBBER OPEN GRADED MECHANICAL PROPERTIES

Arizona State University did extensive mechanical testing of ARFC mixes over the last 10 years (Kaloush et al. 2011). The testing included tests for the triaxial shear strength, dynamic modulus, permanent deformation, beam fatigue indirect tensile strength and creep; fracture and crack propagation; and moisture damage. Based on the asphalt rubber open graded (ARFC) laboratory test results from many studies, the following general observations about their mechanical properties were determined. Because the ARFC mixes have a very high air void content by design they have to be tested in a confined condition to obtain realistic dynamic modulus values. ARFC mixes typically are able to reduce the occurrence of reflective cracking and cracking in general. This crack reduction capability is substantiated by fracture test results, Figure 8 and beam fatigue test results, Figure 9. These test results show that ARFC mixes are better able to resist fracture and fatigue cracking. ARFC mixes are generally do not contribute to rutting since they are only placed 25 mm or less in thickness at the top of the pavement structure where shear stresses are very low. Likewise the single size stone gradation creates virtually only stone to stone contact also which reduces rutting.

Figure 8. Greater total fracture energy with ARFC compared dense graded conventional HMA.
7 CONCLUSIONS

Historically open graded friction courses have been used but oftentimes suffered from a tendency ravel due to traffic and/or wet weather. Starting in the 1980’s asphalt rubber open graded mixes (ARFC) began to be used in Arizona, California and Texas. Since then thousands of kilometers of this type of surfacing has been constructed to provide a good skid resistant surface, smooth riding surface, reduce noise in urban areas reduce reflective cracking. Use of ARFC’s has grown around the world. Portugal in the 1990’s to reduce noise, China in the 2000’s to reduce noise and Brazil in the 2010’s to improve skid resistance and reduce reflective cracking. ARFC’s are a very useful type surface to provide improved skid resistance, improved ride and reduce noise. Based on studies in Arizona, Figure 10 demonstrates such performance improvements can last up to a period of ten years (Kaloush et al. 2011).

<table>
<thead>
<tr>
<th>ARFC 25 mm Overlay Smoothness, Skid and Noise compared to PCCP</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>Smoothness IRI m/km</td>
</tr>
<tr>
<td>Skid at 100 km/hr</td>
</tr>
<tr>
<td>Noise dB</td>
</tr>
</tbody>
</table>

Figure 10. 10 years of better smoothness, skid resistance and noise with ARFC.
8 REFERENCES

**PAPER TITLE** (90 Characters Max)  | **INSTITUTIONAL APPROACH IN STRENGTHENING INDONESIA BRIDGE MANAGEMENT: A LESSON LEARNED FROM COLLAPSED KUTAI KARTANEGARA SUSPENSION BRIDGE**  
---|---
**TRACK**  |  
**AUTHOR** (Capitalize Family Name)  | **POSITION**  | **ORGANIZATION**  | **COUNTRY**
Herry VAZA | Director | Institute of Road Engineering, Ministry Public Works | Indonesia
**CO-AUTHOR(S)** (Capitalize Family Name)  | **POSITION**  | **ORGANIZATION**  | **COUNTRY**
Haruo ISHIDA | Professor | Dept. of Social Systems and Management University of Tsukuba | Japan
**E-MAIL** (for correspondence)  | **herry_vaza@pusjatan.pu.go.id** **ishida@sk.tsukuba.ac.jp**

**KEYWORDS:**
long-span bridge, bridge management system, bridge safety commission, NSGP, progressive collapse.

**ABSTRACT:**

Bridge is very important for road network. Long-span bridge construction in Indonesia had been started in 1990 to address economic development challenges. Many areas were opened by its existence and accelerate the economic activities in the areas, such as Kutai Kartanegara Suspension Bridge that built in 1995-2000 accelerates economic activities of Tenggarong and Samarinda, capital city of East Kalimantan Province.

In 2011 the bridge collapsed in progressive manner. The investigation indicates that the collapse happened due to improper procedures during the maintenance works. The event makes parties and stakeholders involved in bridge planning and construction realize to the importance of compliance for Norm, Standard, Guideline and Procedure (NSGP) in appropriate ways.

This paper discusses institutional approach for the case in term of legally binding of all NSGP to mandatory. Especially under Indonesia recently decentralized governance, it is urgent to establish Bridge Safety Commission in order to control all stakeholders.
Institutional Approach In Strengthening Indonesia Bridge Management: A Lesson Learned From Collapsed Kutai Kartanegara Suspension Bridge

Herry Vaza\textsuperscript{1}, Prof. Haruo Ishida\textsuperscript{2}
\textsuperscript{1}Institute of Road Engineering, Indonesia
Jl. A.H. Nasution No. 264 Bandung 40294
Email for correspondence: herry.vaza@pusjatan.pu.go.id
\textsuperscript{2}University of Tsukuba
Tennodai, Tsukaba-City, 305873, Japan
Email for correspondence: ishida@sk.tsukuba.ac.jp

1. BACKGROUND

Road networks have important role in Indonesia transportation system, and even dominant mode compared to the other transportation mode. It is predicted that in 2014, the total road in Indonesia covers about 400,000 km long. The land transportation serves approximately 90% volume of goods transportation and more than 95% volume of passenger transport.

According to a workshop held in Ministry of Public Work on 24 February 2014, 90% of national road networks and 70% of provincial road networks are in good condition, while for district road networks less than 50% in good condition.

Bridge as part of the road infrastructure importantly contributes to the functionality of the road networks. Moreover, Indonesia as an archipelago consisting of several islands and provinces (Figure 1) with specific site condition requires more bridge infrastructure. So, it is essential to assure all aspects of bridge establishment conform to standards. Based on BMS data in 2007, there are 89,000 bridges (1,060 km) in Indonesia consisting of 54,000 bridges serve district roads and 35,000 bridges serve national and provincial roads, respectively. It means the bridges covering 0.25% of total length of road networks. If we look further on the figures, approximately 16,962 bridges (325 km) serve national roads, 18,038 bridges (335 km) and the rest 54,000 bridges (400 km) serve province and urban roads, respectively (Bintek, 2007).

By its condition 46\% (230 km) are in good condition, 37\% (185 km) minor and moderate damaged, and 17\% (85 km) heavily damaged or failed.

![Figure 1 Map of provincial government](image-url)
Table 1 Distribution of bridge span in Indonesia (Bintek, 2007)

<table>
<thead>
<tr>
<th>Bridge Span (m)</th>
<th>Percentage of distribution (nos)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-20</td>
<td>78%</td>
</tr>
<tr>
<td>20-30</td>
<td>9%</td>
</tr>
<tr>
<td>30-60</td>
<td>9%</td>
</tr>
<tr>
<td>60-100</td>
<td>2%</td>
</tr>
<tr>
<td>&gt; 100</td>
<td>2%</td>
</tr>
</tbody>
</table>

Based on the main span, the distribution of bridges in Indonesia can be seen on Table 1. It is clear that majority (i.e., around 78%) of the bridges are short span (0-20 m) type. Only 2% categorized as long span bridges. However, the need for the construction of long span bridges increase as growth of economy development. Similarly, there are several types of bridges in Indonesia (Table 2) such as culvert, girder, truss, etc. Table 2 shows that girder type is dominant type of bridges with percentage around 69%.

Table 2 Bridge distribution in Indonesia based on type (Bintek, 2007)

<table>
<thead>
<tr>
<th>Bridge Type</th>
<th>Percentage of distribution (nos)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Culvert</td>
<td>17%</td>
</tr>
<tr>
<td>Girder</td>
<td>69%</td>
</tr>
<tr>
<td>Truss</td>
<td>10%</td>
</tr>
<tr>
<td>Other</td>
<td>5%</td>
</tr>
</tbody>
</table>

2. BRIDGE MANAGEMENT SYSTEM

Starting in 1990’s Directorate General of Highways, Ministry of Public Work has introduced Bridge Management System (BMS 92) containing guidelines for bridge works from planning to rehabilitation that can be used by bridge designers, owners, contractors, supervisors and rehabilitation applicators. Generally the BMS’92 is intended to be applied for bridges of National, Province and District roads. But in practice, there are limitations which make the system are not suitable in every case of local government. The system is especially fit for organization already implemented partially for bridges in national and province roads, but not for bridges in districts and cities.

This system is applied for planning and programming of bridge maintenance, where many bridges built in 1970s and some were deteriorated and need proper maintenance. Figure 2 shows a general bridge deterioration model. Figure 2 indicates that bridges require a routine maintenance so that further and worse damages can be avoided. Maintenance could extend service life of bridge. For heavily damaged bridge, rehabilitation may improve the bridge condition.

Road networks management system in decentralized government started from 1999/2000, which separate national road and bridge management. National road and bridges are under responsibility of government, whereas provincial and local roads are under responsibility of Local Government. However, for technical empowerment or development is still under associated technical ministry related with road and bridge, i.e., Ministry of Public Works.
In the decentralization era, Local Government has greater roles in infrastructures, where many roads and bridges were built as an effort to open access and develop the region. Unfortunately, some local governments lack of adequate capacity to implement the responsibility. As consequence, possible disaster may occurs due to inappropriate application.

**Technical Guideline & Design Criteria**

As mentioned briefly in the background, the operation of road infrastructure is necessary to ensure that built infrastructure meets their proper function, including the bridge infrastructure. There are several steps to be followed in road development, as applies in general construction phases, namely Planning, Construction, and Operations and Maintenance. Especially for the Planning stage, there is a regulation by the Minister of Public Works that specifies the technical road planning and substances contained herein.

Based on Law No. 38/2004 on Road and Regulation of Minister of Public Works No.19/2011 on Technical Requirement and Design Criteria, it is stated that the Planning of roads comprises of 3 phases namely: Initial Technical Planning, Feasibility Studies and Final Technical Planning (Figure 3). The three stages are distinguished by the completeness of data collection and review of proposed alternatives. In the Initial Technical Planning, the activities include several alternatives of road alignment to be built, studying and providing technical, economical, environmental and safety considerations of the concept plan of the proposed alignment. The next stage is the Feasibility Study of the road network. The scope of the feasibility study stage covers the study of technical feasibility and financial viability of each alternative alignment proposed in the Initial Technical Planning stage. This stage establishes the alternative choice of alignment that is most technically and financially feasible and also most feasible in terms of road safety.

The final stage of the procedure is Final Technical Planning - final engineering design. At this stage, the supporting data collection activities are implemented in the planning. The intended scope of the data/information collection includes reviews of the field survey to determine the final alignment based on several selected alternatives of the road feasibility study results. The collected information will be technical basis for the detailed planning - detailed engineering design. The resulting detailed technical planning is audited to ensure that the final planning is safe for the prospective users. Results and recommendations of the audit findings are accommodated in the final stages of the technical planning. The stages of road network development are shown in the following figure:
Bridge Asset Framework

Comprehensive bridge asset management documentations were completely available, but may not in detailed. The available guideline from Bridge Management System 1992 (BMS’92) such as Code and Practices as well as Specification including Bridge Inspection Manual and Programming/Programming was sufficient enough for implementing asset management in order to ensure appropriate operation of bridges to support optimum use of road networks. Figure 4 shows Bridge Asset Frameworks activities and the relation to NSGP as well as to Ministry Public Works. For Planning, as have been explained above, Operation and Maintenance, Bridge Inspection modules, and Planning and Programming modules of BMS’92 are already available for determination of bridge maintenance category and strategy.

In addition, Norm, Standard, Guideline, and Procedure (NSGP) have been developed for bridge management system. This NSGP could contribute to the bridge preservation in Indonesia as well as technical guide in the works carried by Ministry of Public Works in accordance with prevailing regulation.

Figure 3 Steps of Road and Bridge Technical Design

Figure 4 Bridge asset frameworks
In addition, based on Article 13 of Law No. 38/2004 on Road, the authority for road structure including bridges and other structure is under the same authority of the road. Government provides authority to Local Governments to manage road networks within their jurisdiction. According to this Law, Ministry of Public Works is responsible for development of road networks, except for operation and traffic management. The latter is under responsibility of Ministry of Communication, or specified authority.

In general, documentation of bridge asset management is still limited for bridges with span less than 100 m. In fact less documentation made for structurally complicated bridges. Thus, in order to manage longer bridges not covered in the documentation, the Government has issued and set up additional policies especially those related to planning stage, namely:

a. Letter from Director General of Highways No. UM.01.03-Fb/242 date 21 March 2008 regarding Design Provision and Revision of Road and Bridge Design and TOR to cite references in the Directorate General of Highways which contains some important points, among others: (a) Preparation and Legalization of Detailed Engineering Design (DED) Documents for Road and Bridges, (b) Design Criteria for Bridges, and (c) Procedures and Authority for Contract Amendment.

b. Letter from Director General of Highways No. UM.01.03-Fb/27.1 date 10 January 2007 regarding Mechanism for Legalization of Technical Planning Document which contains some important points, among others: (a) Document of technical planning prepared by planners and approved by the organizers or appointed officials and technical planners responsible for the full technical plan documents and (b) technical planners must meet the skill requirements in accordance with the legislation in the field of construction services.

c. A policy in the technical stage to carry out an independent checking applies to bridges considered to have a high level of complexity such as arch bridges, cable stayed bridges, relatively long span suspension bridges. The checking carried out by an Independent Consultant to insure the quality of planning. It has been implemented for examples: Suramadu Bridge Project, Merah-Puthi Bridge Project, and Mahkota II Bridge Project.

d. In the implementation stage, some projects were provided with the Technical Team as the counterpart to the project manager to discuss on issues during bridge construction. In particular the team designated for the bridge projects considered as one of high level of complexity such as; arch bridges; cable stayed bridges as well as relatively long span suspension bridges which difficulties may challenge in the implementation.

e. Minister of Public Works has issued Ministry Rule No. 20/2010 regarding Guidelines and Utilization and Use of the Road Sections. This rule provides dispensation for heavy loading and oversize trucks to access road networks for transporting the Goods or Machinery for industry. The provision involves multiple agencies and its application still depends on the commitment of other agencies.

3. KUTAI KARTANEGARA SUSPENSION BRIDGE

In the 1990s Indonesia has begun to develop a long-span bridge construction. This is done to answer for the challenges of economic development. Access to many regions are opened, so more long-span bridges constructed and applied in Indonesia. Adoption of an advanced bridge technology has been carried out such as the use of pre-stressed concrete bridge, cable-stayed bridge and suspension bridge as well as construction of bridges with aesthetic considerations. To comply a standard, many factors may influence the design, construction as well as operation and maintenance stages of long-span bridges. Therefore, government has issued various regulations to minimize those factors with comprehensive standard procedures.

However, the failure of bridge still may occur, either during construction or during operation. It happened to Air Pangi Bridge (South Sumatra), Cipunagara Bridge (West Java), Sicanang Bridge (North Sumatra) as well as Timpah Bridge (Central Kalimantan). The most recent and fatal failure occurred on 26 of November 2011, where Kutai Kartanegara (Kukar) Bridge as one of the longest span bridges in Indonesia suddenly collapsed in progressive manner. The bridge located in Tenggarong City, East Kalimantan connecting Tenggarong and
Samarinda and its management is under authority of Local Government of Kutai Kartanegara. The collapse of the bridge makes Indonesian government realized that the existing procedures are inadequate for construction and operation of the long-span bridges. The lack of control by government responsible for the bridges in term of technical aspect also contributes to the failure (Kukar).

It is clear that the accident has triggered to review all aspects of bridges construction, including procedure and how the implementation at the field under the decentralized era based on the investigation made so far. Figure 5 shows the percentage of contributing factors for the collapse, namely flooding, overload, lack of maintenance, overload or lack of maintenance, mistakes during design and construction stage, and negligence in rehabilitation activities. Most failures occur either during construction or during operation. Based on the data of Bridge Management System the collapse of bridges may happen both on a long and short span bridge in many locations nationwide, provinces, districts, regardless rural roads or even urban roads in the cities.

![Figure 5 Contributing Factors of Bridge’ Collapse in Indonesia](image)

As illustrated in Figures 6 and 7 Kukar bridge was inaugurated on September 21, 2001, in Tenggarong, Kutai Kartanegara. The bridge connects Tenggarong and Samarinda, i.e., capital city of East Kalimantan. In addition to support activities of local economy, Kukar bridge is also a symbol of the people of Kutai Kartanegara.

![Figure 6 Elevation of Kutai Kartanegara Bridge](image)
It is a suspension bridge with a main span of 270 m, and became the longest suspension bridge in Indonesia at that time and a little bit longer then Mamberamo suspension bridge (235 m) in Papua and Barito twin cable catenaries suspension bridge (240 m) in South Kalimantan. On Saturday November 26, 2011, more than 10 years since the inauguration of the bridge, people of Kalimantan were shocked by sudden collapse of the bridge in less than 20 seconds. The connecting traffic between Tenggarong and capital city of Samarinda was immediately frozen after the collapse. The remaining structure of the bridge is shown in Figure 8.

4. DISCUSSION

In relation with the implementation of written procedures on the bridge in Indonesia, Bridge Management System (BMS’92) seems reliable to be used as a reference. As they have been compared to the regulations of other countries beforehand, therefore theoretically, the planning can run well. If there are differences, the results are an error occurred in the implementation phases. On the other hand the quality of the design of the bridge also cannot be guaranteed because it depends on accuracy of the selection and competence of the designer.

The design of the bridge is always handled by the party who are competent in their field. However, this does not rule out the occurrence of negligence. Generally, negligence is not detected quickly, until some physical signs and visuals such as excessive vibration and excessive deflection identified. But, it is often too late to anticipate.
Bridge Design Quality

Caltrans (USA) applies an effective strategy to overcome this problem with the concept of Independent Checker. This concept refers to the second engineering consultant that works as an independent checker, to recalculate the designs from the first consultant design based on the design drawings. It is procedural, so the first consultant realizes that the design will be reviewed again by other professional consultants. This is the way to make the first consultant always come up with a good design.

Pruefingineur (Engineer Examiner, Germany) also apply the same approach. So is it general in the USA to provide the Professional Engineer (PE) for each engineering product.

BMS’92, Ministry of Public Works has a similar concept as Caltrans’s Independent Proof Check. Independent Proof Check requires examination of other parties that is associated with the design that being made. The project which has utilized that concept is Barito Twin Catenaries Suspension Bridge and Suramadu Cable-Stayed Bridge. The selected Independent Proof Check is a world-class engineering consultancy firm, i.e. Chapman and Wagon from England and COWI from Denmark, respectively. IPC for the latest project of Merah-Putih Cable-Stayed Bridge in Ambon, Maluku done by PT Giritama Persada, Indonesia and Associated with Wiecon, Taiwan. Table 3 shows the minimum checking requirements for bridge’s design bridges where the type of evaluation depends on the degree of complexity and novelty of structure concerns.

<table>
<thead>
<tr>
<th>Degree of Novelty</th>
<th>Degree of complexity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Novel</td>
<td>Complex</td>
</tr>
<tr>
<td></td>
<td>Independent proof-check</td>
</tr>
<tr>
<td></td>
<td>Average</td>
</tr>
<tr>
<td></td>
<td>Independent proof-check</td>
</tr>
<tr>
<td>Non-standard</td>
<td>In-house detailed check</td>
</tr>
<tr>
<td></td>
<td>Average</td>
</tr>
<tr>
<td></td>
<td>In-house detailed check</td>
</tr>
<tr>
<td>Standard</td>
<td>Simple</td>
</tr>
<tr>
<td></td>
<td>In-house supervisor check</td>
</tr>
</tbody>
</table>

Notes:
1. The shaded boxes in the table indicate combinations of complexity and novelty which should never occur in practice.
2. The levels of checking shown in the table are the minimum recommended. More extensive design, checking may be necessary, depending on the circumstances.

Bridge Construction & Operational Quality

Based on Law No. 38/2004 on Roads and Law No. 18/1999 on Construction Service, every project must be completed with technical document prepared by competent and certified consultant. For that reason, the consultant is responsible for planning technical document. Refer to the endorsement letter from Directorate General of Highways (DGH) regarding technical legalization of drawing, 2008.

The readiness of planning stage will determine the level of success of bridge construction. The completed and detailed of construction technical documents, the lower risk will be faced. Any imperfection in construction detail document potentially creates a slow failure until catastrophic sudden failure. Bridge failure could also happen due to wrong procedure and environmental condition. If the collapse occurs in the construction stage, it is easy to identify which party is faulted. As they are still in the construction stage, it involves planner, contractor, and supervision, but not the owner.

This condition will be different if the collapse happens several years after Final Hand Over (FHO). If the bridge failure occurs before 10 years, this condition is categorized as a construction failure and each related party could be asked for the liability including the user as they are stated on Law of Construction Service. If the collapse happens after 10 years, the roles of Planner, Contractor, and Supervision related with the bridge construction is not prominent, because there are other factors beside routine maintenance. Several events might trigger bridges’ collapse, including mistakes in bridge maintenance. Figure 9 shows scheme of responsibility of parties on Construction Failure and Collapse during Lifetime of bridge.
In terms of responsibility and the relation to the risk of failure of the bridge construction, it has been
governed by the Construction Service Law 19/1999 and described in Figure 9. The risk level of construction
failure at the planning and implementation are quite high as shown in Figure 10 below and it is easier who
responsible. This is because the involved parties are still there and obviously related, so the improvement can
be done quickly. Except if the construction failures resulting death victim.

![Figure 9 Construction failure and serviced collapse](image)

**Figure 9 Construction failure and serviced collapse**

![Figure 10 Level of risk and cost associated to bridge construction stage](image)

**Figure 10 Level of risk and cost associated to bridge construction stage**

Generally, in Indonesia the problems may occur at each stage of the bridge construction implementation as
they are related to the competence of human resources (HR) involved. Human resources who works in the
bridge projects are less competent and require guidance, while the experts who works in bridge field are still
limited. Problems in the bridge construction are summarized in Table 4.
Table 4 Problems in bridge management activities

<table>
<thead>
<tr>
<th>Programming</th>
<th>Design</th>
<th>Construction</th>
<th>Operation &amp; Maintenance</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Condition of the bridges is not up-to-date due to the existing system that has not been used optimally.</td>
<td>• Lack of human resources development.</td>
<td>• Lack of human resources development.</td>
<td>• Coordination among agencies is not optimal, including the utilization of river pass by the bridge.</td>
</tr>
<tr>
<td>• Number of bridges, and scattered, while the inspectors are still limited.</td>
<td>• Coordination among agencies is not optimal, including the utilization of river pass by the bridge.</td>
<td>• Coordination among agencies is not optimal, including the utilization of river pass by the bridge.</td>
<td>• Consistent policy in bridge management is still not going well.</td>
</tr>
<tr>
<td></td>
<td>• In the new bridge, the investigation data of surveys is not reliable.</td>
<td>• Lack of mastery in construction technology (materials, tools, etc.).</td>
<td>• Lack of Maintenance.</td>
</tr>
</tbody>
</table>

Note: compilation from many sources

Learning From Collapse of Kukar Bridge

The failure case of Kukar Bridge gives a valuable learning for the civil engineer and bridge authority as well as the stakeholders which concern to the bridge construction in Indonesia. In this case, cause of collapse can occur and need to be evaluated at least from detailed engineering stage, construction and imperfection happened to the bridge during service more than 10 years up to the an effort to rehabilitate the elements of bridge’s imperfection.

Based on the case, it indicated that the collapse triggered by any fault of bridge hanger jacking procedure in order to adjust to balance forces between the bridge hanger. Figure 11 shows the bridge performance for the critical elements during service. Imperfection happens at the beginning of commencement of bridge operation. However, it was stable after first year. This normally happens in field of civil construction when the structure achieves new equilibrium as the load was applied. Moreover, for engineering justification before commencement of service, prior to Final Handover (FHO), the bridge was tested through static and dynamic loading test, the result of the performance and load capacity was confirmed to the design.

![Camber Jembatan Kukar](image1)
![Penurunan A1 & A2 Jembatan Kukar](image2)

(a) Camber Jembatan Kukar  
(b) Penurunan A1 & A2 Jembatan Kukar

**Figure 11 Imperfection of bridge critical components and stable after first year in service:** (a) bridge deck camber, (b) bridge anchorage both sides

Prior investigation indicates the causes collapse is a steel material used in bridge clamps. However, after testing in laboratory at structure scale, it was proved that the design load to be confirmed to the specifications.
Moreover, it is clear that the issue of collapse related to Design and Construction stage was not relevant as it does not meet any provision stated in the Construction Service Law, see Figure 9 above. So what does the cause of the accident? It is maintenance works as the main cause of the collapse. Figure 12 shows the maintenance activity when collapse happened. The failure may occur in maintenance works: competence; working environment; and detailed operational procedure as well as the workmanship, probably contributes to the main causes. Figure 13 shows detailed hanger extension for jacking table may sources of the accident. It may happen due to lack of control and lack of knowledge as well as an effort of any maintenance works which do not comply with the provision.

![Figure 12 Maintenance works when accident happen](image1)

![Figure 13 Detailed hanger extension for jacking table](image2)

5. CONCLUSION

Bridges play very important role for road network. Long-span bridge construction in Indonesia had been started in 1990 to address economic development challenges. Many areas were opened by its existence and therefore accelerate economic development, such as Kutai Kartanegara Suspension Bridge built in 1995-2000 that accelerates economic development of Tenggarong and Samarinda, capital city of East Kalimantan Province. In 2011, the bridge collapsed in progressive manner. The collapse indicates due to improper procedures during the maintenance works. This event makes stakeholders realize on the importance of compliance with Norm, Standard, Guideline and Procedure (NSGP). The lack of control for bridge establishment especially in local government level also contributes to the collapse due to shared funding from local government for long span bridges.
It is important to make a legal binding for bridge management so that local government as well as authority for national bridge could consistently comply with the rules and at the same time strengthen their human resources capacity. Finally, in order to control the application at field, it is important to establish such a Bridge Safety Commission at least during the beginning stage of bridge development to ensure and audit the compliance of the rules. Figure 14 shows interrelation within Public Works organization and bridge safety commission in development and maintenance of Indonesian bridges.

![Figure 14 Interrelation within Public Works Organization and Bridge Safety Commission](image)

**REFERENCES:**
2. Asset Management of Road and Bridges, (2012), JICA.
7. Danish Bridge Management System, Danbro, Denmark
8. Decree of Minister of Public Work No.297/KPTS/M/2013 on Task Force for Disaster Mitigation in Ministry of Public Work, 2013, General Secretariat of Ministry of Public Work
11. Direktorat Bina Teknik, Bridge Maintenance Book (Collection), Direktorat Jenderal Bina Marga.
15. Documentation of Bridge Failure in Indonesia, (http://www.google.com/)
16. Documentation of Bridges Administration in Indonesia, (http://www.google.com/)
17. Challenges on the Bridge Management Activities Towards Sustainable Bridge Development In Indonesia, (2013) JRC, Tokyo
Road Safety Assessment of Southern East Java National Corridor

Ir. Herry Vaza¹; Ir. IGW Samsi Gunarta²; Muhammad Idris³
¹Director of IRE; ²Head of Traffic Engineering and Road Environment Laboratory – IRE; ³Road Safety Researcher – IRE; Institute of Road Engineering (IRE) Agency for Research and Development, Ministry of Public Work - Indonesia

ABSTRACT

The target of National Safety Plan on lowering the fatality rate in traffic accidents to become 50% (1.96 fatality /10.000 vehicles) of the 2020 projection has become a challenge to the Ministry of Public Works of Indonesia. To provide an overview of safety levels of the national road, IndII (Indonesian Infrastructure Initiative) in collaboration with the Institute of Road Engineering (IRE) conducted assessment using the International Road Assessment Program (iRAP) tool kits in Southern East Java Corridors (SEJC) of Indonesian national road network. In total, the 416.2 km length has been inspected using the Hawkeye system and assessed using the iRAP online software. The complete results for review are available in the software. Key characteristics of the network are 97% is undivided with 4,098 significant intersections; mostly (88%) in 50 km/h speed zone; and 12% in 80 km/h. Around 65% is straight or gently curving; and 92% has hazards within 1 to 5 meters of the left side of the road, while all of the length has no facilities dedicated for motorcycles.

Star ratings were applied for car occupants, motorcyclists, bicyclists and pedestrians at the prevailing traffic speed. It reveals that from total 416.2 km, safety level for car occupant categorized as 1 to 5 varies between 19 to 22 %. Motorcyclists range between 18 to 22 %, which shows similar fashion with the pedestrian. The cyclist safety level shows more variation (between 18-24%), while overall, the safety level of the road performs 21% of 1-star, 19% of 2-star; 24% of 3-star, 18% of 4-star and 18% of 5-star. To improve road safety level, focuses should be set to road sections with star rating 2 and 1, since they are considered to be high accident risk.

Keyword: road safety performance, road assessment, star rating, international road assessment program (iRAP)

1. BACKGROUND

Fatality index is one indicator of accident assessment which basically gives the size, by utilizing traffic as exposure. Another common size used is the accident rate as well as exposure to utilize AADT unit (acc per 100MVKT). This size can be used on assessing the condition of road safety and the fatality of the national target achievement size index on reducing the fatality rate. The national target as it was programmed in the Decade of Action (DoA) was a 50% reduction in fatality rate projected for 2010 and it is equivalent to 1.96 fatality per 10,000 vehicles. The target of 50% accident fatality rate reduction is an aggregate on achieving the fifth pillar target of DoA which cannot be directly used to assess the performance of each pillar. Therefore, an appropriate performance measure for each pillar is needed.

Recently many star rating models being developed to measure the performance of safety in each pillar of the safe system. One of the star rating methods mostly utilized by various countries is road assessment concept initiated by the International Road Assessment Program (iRAP). The iRAP methods assess road infrastructure using more than 21 attributes from perspectives of four road user types (car occupant; motorcyclists; bicyclists; pedestrians). It is targeting the level of safety of segment or road network performance. The software used in iRAP methods is not only able to issue the performance measure in form of star rating but also including the facility to indicate necessary measures to improve road safety and yield investments plan for road safety improvement program.

As part of the IndII program, road safety improvement of national highway have been considered to be one major activity complimentary to the road worthiness requisites, the star rating system was trialed to be able to provide alternative tools for road safety improvement in cooperation with the IRE. The trial was carried out at national highways under the administration of the Regional Office V of the DG of Highways, which includes Central and East Java Provinces. The 3-month trials (April to June 2013) resulted full assessment of 416.2 Km of national highways mostly in the area of East Java Province. In addition to technical assessment provided by iRAP system, IndII and IRE has also conducted administration evaluation regarding the capacity of road assessment conducted by IRE using its own resources.

This paper aiming to provide an example works carried out and process of iRAP initiation that might be useful for whom to apply iRAP as tool for road safety improvement.
2. STAR-RATING SYSTEM AND ROAD SAFETY IMPROVEMENT

Star-rating approach and road investment plan development are two important elements to make the road improvement toward better safety can be done systematically. The star-rating system can help road managers to indicate safety level of the infrastructure as well as to set up an infrastructure improvement target to the level of safety to be delivered to road users. This system is based on visual inspection resulted in road safety assessment following the protocol shown in Figure 1.

![Figure 1. IRAP protocol](image)

**Attribute Assessment of Road Safety Performance**

The number attributes of road design elements, definitions and categories for each attribute of a given element of road infrastructure in this paper refers to the attributes developed by EuroRAP and AusRAP. Meanwhile, the risk to the research associated with the infrastructure also refers to EuroRAP and AusRAP. Overall there are 21 attributes of roads that have been used in the iRAP. The attributes were adjusted to the need of road users including car occupants, motorcyclists, cyclists, and pedestrians. Table 1 shows each attribute for every road user.

<table>
<thead>
<tr>
<th>Road Attribute</th>
<th>Type of Influenced Road Users</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Car occupant</td>
</tr>
<tr>
<td>1. Bicycle Facility</td>
<td>✓</td>
</tr>
<tr>
<td>2. Delineation</td>
<td>✓</td>
</tr>
<tr>
<td>3. Intersection Volume</td>
<td>✓</td>
</tr>
<tr>
<td>4. Intersection Type</td>
<td>✓</td>
</tr>
<tr>
<td>5. Lane Width</td>
<td>✓</td>
</tr>
<tr>
<td>6. Median Type</td>
<td>✓</td>
</tr>
<tr>
<td>7. Minor access point density</td>
<td>✓</td>
</tr>
<tr>
<td>8. Number of Lane</td>
<td>✓</td>
</tr>
<tr>
<td>9. Passing demand</td>
<td>✓</td>
</tr>
<tr>
<td>10. Shoulder Width</td>
<td>✓</td>
</tr>
<tr>
<td>11. Pedestrian Crossing Facility</td>
<td>✓</td>
</tr>
<tr>
<td>12. Pedestrian Crossing Quality</td>
<td>✓</td>
</tr>
<tr>
<td>13. Curve Quality</td>
<td>✓</td>
</tr>
<tr>
<td>14. Intersection Quality</td>
<td>✓</td>
</tr>
<tr>
<td>15. Bend Radius Curve</td>
<td>✓</td>
</tr>
<tr>
<td>16. Pavement Condition</td>
<td>✓</td>
</tr>
</tbody>
</table>
Road Protection Score

After inspecting of road elements, road protection value or Road Protectors Scores (RPS) was calculated for each 100m of roads using iRAP software. RPS is an objective measure against the possibility of injuries and their severity, based on road infrastructure elements. RPS forms part to produce a star rating. RPS iRAP has been developed from Euro RAP models which assess passenger car protection through road elements in the event of an accident. From AusRAP model that assesses both the protection afforded to passenger vehicles as well as the possibility of accidents. RPS iRAP was also described an extensively research on the risks of accidents related to road infrastructure.

iRAP important aspect in developing RPS is the assessment consistency which can be applied in various countries. Therefore, assessment forms designed by iRAP are directed to provide similar assessments in all countries to predict mortality and serious injury that may occur on a road network. This form provides the basis for estimating the number of deaths and serious injuries to determine the treatment that may be used. Details infrastructure require the development of comprehensive models that include assessing the risk to road users, reported on the proportion of accidents, and implement the details of the relative risk factors. The following is one of the RPS calculations model of car occupant.

The measure of the RPS value every road user is determined from the total RPS calculation of each possible accident types. While the RPS of each accident likelihood type determined by multiplying the risk accidents phi (RPSi) with the severity of accidents (Sevi) and collision calibration factor (CF) as given in equation (1):

\[ RPS_i = |RF_{Li} \times Sevi \times CF | \]  

(1)

RF\(_{Li}\) is an accident risk factor of each likelihood factor to road users. This risk factor is determined from the risk accident model that has been developed by the IRAP. While Sevi is the severity of the accident in terms of the most associated factors with an accident, this factor is also obtained from the various models that have been developed by the IRAP. For the RPS calculation of the passenger vehicle, there is typically considered accident, that is the front-front accident (head-on), out of the road accident (run-off), and accidents at intersections.

Then CF is the calibration factor derived from the available accident proportion. However, when the accident data less qualified, then the iRAP software automatically uses a certain value determined from the accident type that may occur. Equation (2) further illustrates the calculation value of the road protection of road users (RPS\(_{ai}\)) were determined from each sigma protection value of each typical accidents that may occur to the certain road users (RPS\(_i\)).

\[ RPS_{ai} = \sum_i RPS_i \]  

(2)

Below is one of the RPS calculations model for car occupant

RPS for Car Occupant

As shown in Figure 2, the calculation of the RPS for public transport is essentially determined by the value of the accident risk (RF\(_{Li}\)) of each influential factor associated with severity (Sevi) and corresponding collision calibration factor (CF). Thus, the value of protection of each accident type for public passengers is given as follows:

RPS to crash out of the road (Ro: run-off)

\[ RPS_{Ro} = |RF_{Liro} \times Sevi_{Ro} \times CF_{Ro} | \]  

(3)

RPS for Ho (head-on)

\[ RPS_{Ho} = |RF_{Lihfo} \times Sevi_{Ho} \times CF_{Ho} | \]  

(4)
RPS for In (Intersection)

\[
RPS_{In} = \left( RF_{ihn} \times Sev_{iIn} \times CF_{iIn} \right)
\]

RPS for car occupant vehicle road users is determined from the summation of each RPS related accident are given as follows:

\[
RPS_{Co} = RPS_{Ro} + RPS_{Jo} + RPS_{In}
\]

![Figure 2. Calculation scheme of RPS car occupant (source: iRAP)](image)

**Star rating**

Road safety star rating is a measuring tool of accident probability and severity that may occur on a stretch of road. Star rating uses data from road safety as well as the relationship between the attributes and the road accident rate. By measuring the risks associated with the road attribute, star rating may provide a better indicator of the road attribute at risk effect rather than the number of accidents. Star ratings provide a simple and objective measure of the safety level of a given road infrastructure.

**Investment plan**

The goal of safer roads investment plan is to provide a treatment of economics appreciation of a risky reduction. IRAP has considered more than 70 options in a range of treatments from the lowest cost to the highest cost which can be seen in the IRAP toolkit.

In the road investment plan calculation specified several necessity treatment options to improve road safety performance. Treatment options generated by the safety trigger as a trigger of various treatment types on the elements that have safety problems, in other words which form of road element trigger has a low RPS value. The size of the RPS value will affect star rating.

IRAP model assesses the benefits of an applied safety handling on the road network aimed for the fatalities and serious injuries prevention through economical handling. For this purpose, firstly need estimation of fatalities and serious injuries number of each segment in road network. The road length network commonly used by IRAP in the calculation of the victims estimated number about 416.2km and assessment is performed every 100 meters, so that the road network is considered to have 4,162 parts. The number of fatalities in a road network is a combination of all road users who are victims.
The number of severe casualties on each part section (100 m) based on a comparison of 10 serious casualties for any loss of life or death (10:1). This comparison can be seen on The True Cost of Road Crashes: Valuing Life and the cost of a serious injury. While the overall number of fatalities and serious for the entire segments are the sum of each part sections (100 m).

Model of iRAP calculations calibrated using fatality factor, it is intended to ensure that the fatalities estimated number on the segment considered equal to the actual number that occurred in the segment. The actual number of fatalities each year on the determined segment, using the number of occurring fatalities and serious injuries report and the report of the percentage of casualty’s car user, motorcycle, pedestrian street/pedestrian cross and bicycle. However, for the countries that do not have reports or a record of complete accident, can then used an expert estimation or other indicators.

Once the model calibrated the estimation of fatalities number and serious injuries can be made to any part of segments, the results of these calculations are helpful to reduce the number of victim and the determination of the appropriate treatment for a given location (the segment). For the selection of a treatment, there are several requirements of an occurred condition in the segment part that will be handled or referred to the name of the trigger, the trigger is based on the determination of star rating or RPS, road conditions and traffic volume.

Roads that have a high risk assessment and low star ratings should be prioritized for action plans and road safety investment. Roads with low risk assessment and high star rating are associated with little danger for road users and should be low on the priority list. Roads either high risk assessment or high star rating, low risk assessment or low star rating, usually require further investigation. It may also be the site where there should be a review of management both speed and speed limit policy.

iRAP has more than 70 treatment options to overcome the problems of road safety. One type of problem path element (trigger safety) may have several treatment options. Several options have a different impact on the increase of the star rating. The choice is depend on the improvement targets required by the organizers and the availability of funds. For overall improvement of the road network target, the options can be set in packets treatment options. The amount of the handling fee is calculated based on the unit price of the local conditions.

In the countries where in accident data or the data does not meet the required accident in risk mapping can perform fatality estimation. Fatality estimation is a model of crash assignment on the street/road network based on the RPS value and traffic volume. Calculation started for each segment and ultimately added together to obtain the estimated value of the network.

The next stage is the economic analysis. In this stage, a comparison between the different determined packet handling options based on the safety trigger. For each packet handling options, RPS value can be calculated based on the handling. This can be done because it will change the handling situation and road elements condition for the better ones, so that the RPS value will automatically change. After the RPS values estimated fatality rate, the cost of accidents is also calculated based on the new conditions. To be able to determine and compare the economic value derived from each packet handling options, the benefit cost ratio calculated value and the present value is calculated.

Until recently, safer road investment plans have been used in the safety improvement for the poor countries as well as developing countries, and has saved more than 50,000 deaths and serious injuries per year or about 1.2 million dollars in saving. Unlike the star rating that describe relative risk of a road segment, road safer plant investment identify ways in which the star rating can be increased with cost-effective.

3. METHODOLOGY

Road Inspection

The road section star rating process begins with data collection. Generally, data collection techniques performed through visual road inspection and focused on road infrastructure elements. Currently, iRAP uses two road inspection methods where one of them used a survey vehicle, Hawkeye-2000.

Once the video data collected, the assessment conducted through a desktop inspection of road infrastructure by connecting elements of the road network at the time of inspection. The assessment carried out by using special software to obtain accurate elements measurement such as lane width, shoulder width, the distance between the road edges, hazardous locations, and so on.
**Data coding and ViDA Analysis**

Data coding using iRAP method must needs a software that is compatible with the software of the entire recording inspection data. Forms used in this codification process is in digital form which is designed in a multiple choice format.

To conduct the data codeification, coder or rater must use required manual coding of iRAP Star Rating Coding Manual - Drive. This manual is available and can be downloaded on the iRAP website. Coder or rater carrying out the appropriate encoding attribute specified in the Manual Star Rating Coding and should be fit with the specifications of the software. The encoding process performed every 100 meter intervals along the road network.

![Data Coding](image)

![Star rating](image)

![Invest Plan](image)

**Figure 3. Road assessment program step**

Road infrastructure safety analysis was carried out using ViDA software, the web-based software that equipped with the road safety calculation models. Implementation analysis performed automatically in ViDA, a data processing results firstly examined the completeness or quality assurance conducted by QA (quality assurance) personnel who are then uploaded into the ViDA software for each road segments. The results of the ViDA analysis then downloaded in the form of graphs and tabulations road safety conditions of a star rating that describes the elements effect on the potential accidents and the severity that may arise. Star ratings provide a simple and objective measure of the safety level of a given road infrastructure.

**4. IRAP IMPLEMENTATION OF SOUTHERN EAST JAVA CORRIDOR**

**Key characteristics**

The following is the characteristics of road link between Lumajang-Banyuwangi located on the national road corridor of southern East Java. This segment has 416.2 km long, majority (97%) an undivided segments with the average annual traffic flow ranged 15,000 to 20,000 vehicles. Overall 59% of road conditions relatively good, 33% of road conditions categorized average, and the rest (7%) classified as poor.

Geometrically, 416.2 km along the segment consists of a straight lane (65%), sharp turns (20%) and very sharp (1%). This segment has a slope of 0% -7% by 93% and 7% -10% slope of 7%. Nearly half of the road segments width greater than 3.25m; 30% of lane width between 2.75m-3.25 m and 19% less than 2.75 m. Meanwhile, the ideal shoulder width considered only 2% (1.00 m - 2.40 m); The relatively narrow 33% less than 1.00 m; and 65% do not even have the road shoulder.

This road link does not have facilities for bicycles and motorcycles as well as for pedestrians along the roads. Road crossing facilities recorded only 1% (31 points) classified as good quality; 6% (269 points)
considered poor and 93% (3862 points) not applicable. Limitations of this facility is considered very risky against a vulnerable road user group, since 88% of vehicles have a speed of 50 kph and 12% of vehicles have a speed of 80 kph to 85% -tiles speed. Type of median, 95 % shows the centre line road marking median; 2% in the form of physical median.

The land used along the road, eg. Commerce is the more dominant use of the land along the road (40% -42%) and follows residential (37% -40%), undeveloped areas (9%-13%); farming and agricultural (6%), and education (2%). Road side severity along the roads classified as the risk is 7% with a range from 0 to 1.0 meter; 92% is considered moderate with distances range from 1m-5m, and the safest recorded at 1% with distances range between 5m-10m. While the side object type varies including tree (> 10cm dia) (62%); sign post or pole (> 10 cm dia) (27%), non-frangible structure (bridge or building) (6%), etc.

Road rating
Star ratings based on road inspection data and provide a simple and objective measure level of safety which is developed to the road for car occupants, motorcyclists, bicyclists and pedestrians at the prevailing traffic speed. The 5-star (green) roads is the ultimate safest level while 1-star (black) road is the least safe. The overall star ratings for the network is shown in the Table 2 and the typical star rating map for car occupant is shown in Figure 4.

Table 2. Road star rating for four road user

<table>
<thead>
<tr>
<th>Star Ratings</th>
<th>Vehicle Occupant</th>
<th>Motorcycle</th>
<th>Pedestrian</th>
<th>Bicycle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Length (kms)</td>
<td>Percent</td>
<td>Length (kms)</td>
<td>Percent</td>
</tr>
<tr>
<td>5-Star</td>
<td>0.6</td>
<td>0%</td>
<td>0.6</td>
<td>0%</td>
</tr>
<tr>
<td>4-Star</td>
<td>134.9</td>
<td>32%</td>
<td>102.4</td>
<td>25%</td>
</tr>
<tr>
<td>3-Star</td>
<td>137.0</td>
<td>33%</td>
<td>141.3</td>
<td>34%</td>
</tr>
<tr>
<td>2-Star</td>
<td>70.3</td>
<td>17%</td>
<td>67.5</td>
<td>16%</td>
</tr>
<tr>
<td>1-Star</td>
<td>70.7</td>
<td>17%</td>
<td>101.4</td>
<td>24%</td>
</tr>
<tr>
<td>Not applicable</td>
<td>2.7</td>
<td>1%</td>
<td>2.7</td>
<td>1%</td>
</tr>
<tr>
<td>Totals</td>
<td>416.2</td>
<td>100%</td>
<td>416.2</td>
<td>100%</td>
</tr>
</tbody>
</table>

Figure 4. Road star rating map for car occupant

Table 2 shows that the 1-star rating ranged between 17%-33% for all road users; for a 2-star rating varies between 17% -47%; for a 3-star rating varies between 33-40%; 4-star rating ranged from 25%-32% just for car occupant and motorcycle; 5-star rating and recorded only 0.6% for the motorcycle. The table further shows that the 1-star rating for the vulnerable road user groups considered still quite high (23% -33%). Similarly to the 2-star rating was also higher for bicycle (22%) and pedestrian (47%). From this table indicates that pedestrian facilities and bicycles almost nothing, in other words that the road is very risky to pedestrians and bicycles group.
**Countermeasures - Investment Plan**

Table 3 shows that the investment plan for the road safety quality improvement with the proposed 15 priorities handling generated by ViDA software analysis. Road investment plan as in Table 3 is equipped by a number of important information including information of length/site, FSIs saved, PV saved benefit, estimated cost, cost per FSIs Saved, and CB-ratio. The original table of the investments plan contains more than 60 types of countermeasures. Overall if the countermeasure option is implemented it will provide FSIs-saved value of 302, CB-ratio 17 and the total estimated cost of over IDR 5 trillion and IDR 93 trillion benefits.

The best choice of investment plan is based on the value of the highest CB-ratio, the highest FSIs-saved, and the lowest estimated cost. Fifteen types of countermeasures in Table 3 is a priority program with the highest CB-ratio as well as the lowest estimation cost. Pedestrian fencing, for example, has the highest BCR value (366), cost estimation (IDR. 736,405,020) and high benefit (IDR. 269,245,132,278). These countermeasures are expected to reduce the potential accidents and save 860 pedestrians.

**Table 3. Countermeasures with 15 program BCR priorities**

<table>
<thead>
<tr>
<th>Countermeasure</th>
<th>Length/site</th>
<th>FSIs saved</th>
<th>PV saved of safety benefit</th>
<th>Cost Estimated</th>
<th>Cost per FSIs saved</th>
<th>Program BCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pedestrian fencing</td>
<td>73.3km</td>
<td>860</td>
<td>269,645,132,278</td>
<td>736,405,020</td>
<td>847,647</td>
<td>366</td>
</tr>
<tr>
<td>Improve curve delineation</td>
<td>134.4km</td>
<td>1,720</td>
<td>536,043,327,002</td>
<td>4,278,217,640</td>
<td>2,476,636</td>
<td>125</td>
</tr>
<tr>
<td>Skid resistance</td>
<td>8.8km</td>
<td>2,790</td>
<td>8,593,607,938,784</td>
<td>172,638,983,450</td>
<td>6,233,933</td>
<td>50</td>
</tr>
<tr>
<td>Central full turning lane</td>
<td>7.2km</td>
<td>42</td>
<td>13,126,802,107</td>
<td>445,090,170</td>
<td>10,521,748</td>
<td>29</td>
</tr>
<tr>
<td>Street lighting (intersection)</td>
<td>405sites</td>
<td>330</td>
<td>104,523,549,526</td>
<td>4,572,840,000</td>
<td>13,576</td>
<td>22.9</td>
</tr>
<tr>
<td>Shoulder rumble strip</td>
<td>29.6km</td>
<td>230</td>
<td>72,987,181,242</td>
<td>3,398,131,350</td>
<td>14,447,489</td>
<td>21</td>
</tr>
<tr>
<td>Motorcycle lane (paint on road)</td>
<td>0.3km</td>
<td>1</td>
<td>465,484,618,68</td>
<td>23,981,990</td>
<td>15,927,438</td>
<td>19</td>
</tr>
<tr>
<td>Roadside barrier</td>
<td>46.9km</td>
<td>940</td>
<td>294,281,994,543</td>
<td>18,324,715,500</td>
<td>19,322,902</td>
<td>16</td>
</tr>
<tr>
<td>Central haching</td>
<td>0.4km</td>
<td>1</td>
<td>178,774,856</td>
<td>12,194,250</td>
<td>21,166,425</td>
<td>15</td>
</tr>
<tr>
<td>Sight distance</td>
<td>1.2km</td>
<td>21</td>
<td>6,781,345,496</td>
<td>487,770,050</td>
<td>22,320,199</td>
<td>14</td>
</tr>
<tr>
<td>Improve delineation</td>
<td>184.3km</td>
<td>420</td>
<td>132,507,235,501</td>
<td>9,295,154,570</td>
<td>21,767,865</td>
<td>14</td>
</tr>
<tr>
<td>Central median barrier</td>
<td>0.5km</td>
<td>22</td>
<td>6,926,671,460</td>
<td>544,061,700</td>
<td>24,573,752</td>
<td>13</td>
</tr>
<tr>
<td>Pedestrian facility</td>
<td>156sites</td>
<td>81</td>
<td>25,367,925,457</td>
<td>2,014,309,650</td>
<td>24,639,965</td>
<td>13</td>
</tr>
<tr>
<td>Lane widening (&gt;0.5m)</td>
<td>5.5km</td>
<td>300</td>
<td>94,934,197,228</td>
<td>12,388,000,000</td>
<td>40,492,755</td>
<td>8</td>
</tr>
<tr>
<td>Side road unsignalised pedestrian crossing</td>
<td>38sites</td>
<td>39</td>
<td>12,328,910,299</td>
<td>1,531,055,990</td>
<td>38,535,871</td>
<td>8</td>
</tr>
</tbody>
</table>

5. **DISCUSSION**

Basically investment plan contained numbers of countermeasures constitute ViDA analysis result towards environmental assessment of road conditions and road environment of the four perspectives of road users. This investment plan was designed to improve road safety conditions in the form of star rating. Countermeasures options adjusted to the availability of the budget. Even the results of the investments plan can be directed to the program and increase safety planning in accordance with budget availability. However, the overall results of the investments plan to the value saved by FSIs-302 and CB-17 ratio has provided significant improvements to increase the star rating of the road segments along the 416.2km. The following are there view of a number of proposed countermeasures for CB-ratio programs above 17.

**Pedestrian fencing;** the number of proposed treatment of potential pedestrian accident spread along the roads. From the analysis results as it was presented in the key characteristic description shows that the lack of pedestrian facilities along the 416.2 km of analyzed road link. The typical land use along the road segment dominated by commercial and residential areas which have problems with pedestrians. To improve road safety situation, pedestrian fencing along the 73.3 km is needed. The highest CB-value ratios (366) with the cost value per FSI saved is relatively low compared to other countermeasure which is a priority handling and give significant impact on improving safety conditions.

**Improve delineation curve;** the low curve quality which only 3% was considered adequate, and unavailability of good delineation along the sites, a special kind of countermeasures on the curve to be a priority. Improving delineation curve becomes a second priority with the results of relatively high CB-ratio.
(125) with the cost value per FSI saved relatively low. These countermeasures are needed, which are located along the 134.4 km spread along the 416.2 km road link.

**Skid resistance:** Visually, pavement condition classified as good, however, the Hawkeyes data shows that there are problems on skid resistance along the 88.8 km. Skid resistance improvement became the third best option generated by ViDA analysis. This was indicated by relatively high CB-ratio (50) and cost value per FSI relatively low. Nevertheless, relatively high estimation cost provides relatively high benefits as well. Improving road surface condition even need a high cost but have a significant impact on improving the quality of road safety.

*Full central turning lane:* site preparation for a full rotation direction using a central turning lane became the fourth best choice based on the results of ViDA analysis. This fact is shown by the relatively high CB-ratio (29) and cost value per FSI relatively low. This necessity spread along 7.3 km with a very low estimation cost. Theoretically, the results of this analysis provide a significant impact on improving road safety quality. Notwithstanding, the rotation direction of this facility needs to be adjusted to the road width and adequate sight distance and must be protected from the road access at this location.

**Street lighting for intersection:** proposed street lighting as countermeasure, especially at the intersections consider by ViDA’s analysis results. ViDA’s analysis shows the types of countermeasures CB-value ratio above the average (22) and the value of cost per FSI relatively low and considered a significant impact on road safety quality. Countermeasure is considered very reasonable to the quality of the analyzed intersections only 1% is adequate, it is estimated that the condition of the existing intersection has a high risk of accidents.

**Shoulder rumble strips:** the fact shows that only 2% of the ideal road shoulder width, 33% of the narrow road shoulder width and side roads do not have shoulders. Meanwhile ViDA analysis shows that the needs of shoulder rumble strips along the 29.6 km (7% of 416.2 km) with CB-ratio is above the average (19) as well as the value of the cost per FSI are relatively low. Although the proposed countermeasures have a good advantage, the availability of adequate shoulder to implement this type of countermeasures should be considered.

**Motorcycle lane:** the key characteristic results showed that the three groups of vulnerable road users are basically unfacilitated along 416.2 km. ViDA analysis further shows that the provision of motorcycle lane along 0.3 km worth considering since these countermeasures have a CB-ratio above the average (19) as well as the cost per FSI values are relatively low.

There are a number of countermeasures that could be considered in addition to the six highest CB countermeasures ratio. When countermeasures with CB-high ratio may be applied not ideal, other countermeasures with CB-lower ratio can be considered. It should be underlined that a number of countermeasures generated by ViDA analysis are the default program of iRAP ViDA Software. Several types of countermeasures available in the ViDA software is not all fit with field conditions and should be adapted to the geometric and technical specification in Indonesia.

6. **CONCLUSION**

- iRAP is a road safety management system based on the infrastructure road condition assessment by utilizing a number for assessment criteria. As an assessment program, IRAP provides an assessment of road safety conditions evaluated from four road user perspectives which is then presented in the form of star rating. The results of this assessment are considered easier for policy makers and planners in improving road safety conditions.
- Besides providing safety performance of road infrastructure assessment, iRAP with ViDA analysis software provide approximately 70 handling options through road investment plan which can be utilized to improve the star rating.
- The number of treatment priorities of existing countermeasures is outline in a road investment plan based upon the high FSiS saved, high PV saved benefit, low estimated cost, low cost per FSiS Saved as well as CB high ratio.
- Based on the experiences of various countries; implemented IRAP provide significant results on improving the road safety quality. The concept of road performance evaluation through IRAP approach is considered very helpful to be implemented in Indonesia.
7. ACKNOWLEDGEMENTS
The author would like to thank Greg Smith, Director of iRAP Asia Pacific; Luke Rogers and MIROS staff who involved for implementation iRAP in Indonesia; and special thanks to Road Safety Research members of Institute of Road Engineering who dedicated for development of iRAP in Indonesia.

8. REFERENCE
ADAC, UK Trial Survey to establish Road Protection Scores for the UK Final, 2007
AusRAP, 2008, Star Rating for Queensland Country Highways, Traffic & Safety Department, RACQ
Dahdah S, Safety Rating of Road Infrastructure-Focus on Vulnerable Road Users, Road Safety Learning Day, 30 March 2007
D Lynam, Development of Risk Models for the Road Assessment Programme, TRL, February 2012
IRAP, 2009 (b), The Irap Methodology: Safer Roads Investment Plans, International Road Assessment Programme, London
IRAP TOOLKIT, http://toolkit.irap.org/
International Road Assessment Program (iRAP), Vehicle Speeds and the iRAP Protocols, Policy Position, iRAP 2010
International Road Assessment Program (iRAP), Star Rating Inspection Manual, Setting the standards for the road rating process, June 2010
International Road Assessment Program (iRAP), The iRAP Working House Workshop 2010, Review of the iRAP Road Protection Score model and Star Ratings, 12-13 May 2010 Basingstoke, UK
International Road Assessment Program (iRAP), Establishing iRAP In Your Country, United Kingdom, International Road Assessment Program (iRAP).

***
**PAPER TITLE**

Construction and Operation of SMART Highway Test Bed

---

**TRACK**

Integrated Mobility & Intelligent Transportation Systems

---

<table>
<thead>
<tr>
<th>AUTHOR (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joonsoo SHIN</td>
<td>Senior Manager</td>
<td>Korea Expressway Corporation</td>
<td>South Korea</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CO-AUTHOR(S) (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eui-Joon LEE</td>
<td>Managing Director</td>
<td>Korea Expressway Corporation</td>
<td>South Korea</td>
</tr>
<tr>
<td>Sug-Tae KIM</td>
<td>Chief Research Administrator</td>
<td>Korea Expressway Corporation</td>
<td>South Korea</td>
</tr>
</tbody>
</table>

**E-MAIL (for correspondence)**

halfwing27@ex.co.kr

---

**KEYWORDS:**

SMART Highway, Test bed, Demonstration, V2X, WAVE Communication

---

**ABSTRACT:**

Smart Highway is a national R&D project which started in 2007 and test bed for this project has performed since 2013. Test bed located from Seoul T.G. to Suwon I.C.(111km / 6~8 lanes) on Gyeongbu Expressway.

Test bed will be constructed to correspond for international standards and to induce the development of various applications. To this end, various technologies will be installed on the test bed such as SMART-I (automatic accident detection system), road information detection radar, WAVE road side equipment, SMART terminals, WAVE tolling.

After constructing the test bed, users who install SMART terminal in their car can be offered safety messages throughout V2I and V2V of WAVE communication system. We expect to prevent from car accidents and traffic volume concentration by providing safety messages and vehicle travel information to users.

Finally, we have plans to perform large-scale demonstrations for people on the test bed and a 24-hour operation.
Construction and Operation of SMART Highway Test Bed

Joonsoo SHIN
Senior Manager, Korea Expressway Corporation
50-5, Sancheok-ri, Dongtan-myeon, Hwaseong-si, Gyeonggi-do, Korea, 445-812
+82-31-371-2777, +82-31-371-2779, halfwing27@ex.co.kr

Eui-Joon LEE
Managing Director, Korea Expressway Corporation
50-5, Sancheok-ri, Dongtan-myeon, Hwaseong-si, Gyeonggi-do, Korea, 445-812
+82-31-371-2770, +82-31-371-2779, lejlej@ex.co.kr

Sug-Tae KIM
Chief Research Administrator, Korea Expressway Corporation
50-5, Sancheok-ri, Dongtan-myeon, Hwaseong-si, Gyeonggi-do, Korea, 445-812
+82-31-371-2771, +82-31-371-2779, kst@ex.co.kr

1. INTRODUCTION OF SMART HIGHWAY

SMART Highway Research and Development Project is a national R&D project for developing future road traffic technology aiming to realize safe and convenient highway by combining and fusing advanced road, communication and automobile technologies.

The goal of SMART highway project is to realize fast, convenient and intelligent road by converging IT technologies which is a core value for the road.

Project periods are approximately 7 years(2007.10–2014.12) and 85 million dollars have been invested during those of periods.

2 SMART HIGHWAY TEST BED

2.1 Background of constructing the test bed

The background of constructing the test bed is to verify SMART Highway development technologies on the real road.

A. To correspond the international standards through the availability verification of developed SMART Highway technologies
B. To induce the development of various applications through developed SMART Highway technologies
C. To verify the effectiveness of SMART Highway services(safety messages etc.) to users on real road

2.2 Applied SMART Highway technologies on the test bed

Test bed located from Seoul T.G. to Suwon I.C. on gyeongbu expressway. The length is 11 km and 6–8 lanes with shoulder lane. It is managed by the Korea Expressway Corporation.

Various technologies will be installed on the test bed such as SMART-I(automatic accident detection system), road information detection radar, WAVE road side equipment, SMART terminals, WAVE tolling.

A. Automatic Accident Detection System(SMART-I)
We developed the Array camera to detect incident through composition of panoramic images for 1km section image data by combination of seven cameras and radar and we could overcome the limits(Detection distance : Maximum 200m, error of perspective image) of the existing image detection.
B. Road Information Detection Radar
We developed the radar system to detect fallen objects, surface information such as freezing on road, traffic situation by using radar technology.

C. WAVE Road Side Equipment (WAVE RSE)
We developed the open platform SMART RSE for seamless communication service using WAVE communication technology. It is possible to accommodate various communication systems (Wi-Fi, DSRC, WAVE)

D. SMART Terminal
We developed the SMART terminal for seamless communication service using WAVE. It can incorporate various communication methods. (Wi-Fi, DSRC, WAVE)

Table 1. Technologies and Criteria

<table>
<thead>
<tr>
<th>Division</th>
<th>Qty</th>
<th>Criteria Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Automatic Accident Detection System</td>
<td>2</td>
<td>Tech. Unforeseen accident detection and information collection</td>
</tr>
<tr>
<td>(SMART-I)</td>
<td></td>
<td>Criteria Detecting range (1 km), Traffic accident-prone and chronically congested section</td>
</tr>
<tr>
<td>Road Information Detection Radar</td>
<td>4</td>
<td>Tech. Unforeseen accident detection and information collection (bad weather conditions – snow and rain)</td>
</tr>
<tr>
<td>WAVE Road Side Equipment</td>
<td>9</td>
<td>Tech. Transmission and reception of data and of information: Vehicle-Infra, Infra-SMART Highway Center</td>
</tr>
<tr>
<td>SMART Terminal</td>
<td></td>
<td>Safety 60 - Terminal for safety (Vehicle to Vehicle, Vehicle to Infra)</td>
</tr>
<tr>
<td>Safety +Tolling</td>
<td>40</td>
<td>- Terminal for safety and tolling</td>
</tr>
<tr>
<td>WAVE Tolling</td>
<td>4</td>
<td>- Toll collecting system through nonstop/one lane based on WAVE communication technology</td>
</tr>
<tr>
<td>SMART Highway Center</td>
<td>-</td>
<td>- Construction in Traffic Information Center in consideration of operation management and public relation.</td>
</tr>
</tbody>
</table>

Table 2. Integration of V2X communication test and service on the test bed

<table>
<thead>
<tr>
<th>Division</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Site</td>
<td>Gyeongbu Expressway Seoul T.G. – Suwon I.C.</td>
</tr>
</tbody>
</table>

**Test Goal**
- Test of V2X communication on real road
  - Verifying technologies with many vehicles, obstructions and other wireless communications
  - Integrated test of service

**Test Description**
- Test of V2X communication: communication performance measure in various environment
  - Unit test of SMART Highway service
  - Integrated test of SMART Highway service
  - Test of V2X communication
    - Chain Accident Prevention Service
    - Approaching Emergency Vehicle Warning Service
    - Car Breakdown Warning / Emergency Call or SOS Service
    - Self Vehicle Information Guide Service
3. UTILIZATION OF TEST-BED

3.1 Demonstration of technologies

We will conduct the demonstration to verify and promote SMART Highway technologies. Demonstration of Technology will divide into ‘operation center demonstration’ and ‘demonstration in vehicle’.

Operation center demonstration

We will introduce the SMART HIGHWAY and SMART-I Service demonstration and vehicle control technology based on road information in the traffic information center of Korea Expressway Corporation.

Demonstration in vehicle

Various services based on V2X communication will be provide such as Chain Accident Prevention Service, Approaching Emergency Vehicle Warning Service, Emergency Call or SOS Service, Self Vehicle Information Guide Service, Event Share Service, Construction Zone Warning Service, WAVE Tolling Service, In-vehicle Signage Service on test bed.

- Chain Accident Prevention Service: This service prevents from chain accident (sudden stop, decelerate, broken) by sending the warning message through the V2V communication to following vehicles and near RSEs for a brief intervals of time automatically.
- Approaching Emergency Vehicle Warning Service: When the emergency vehicle approaches for rescue, informing vehicles and RSEs around emergency vehicle of the situation.
- Emergency Call or SOS Service: When the driver pushes the emergency button, the warning message is transferred to the other approaching vehicles and RSEs around disabled vehicle.
- Self Vehicle Information Guide Service: This Service supports a safe driving through that vehicle transfers the vehicle state information (break, speed etc.) to SMART Highway Center through V2I & V2V communication.
- Event Share Service: Prevent from the traffic accident which is happened by the obstacle on the road. The radar detects the situation and sends the message included warning information to the other vehicle in real time.
- Construction Zone Warning Service: The warning information related the road construction is transferred through the SMART terminal from the construction site to the vehicle.
- WAVE Tolling Service: The electronic toll collection and efficiency of traffic and safety service are operated syntagmatically through the high pass service which is applied with WAVE communication.
- In-vehicle Signage Service: This service provides the driver with the customized traffic information based on the location and direction of vehicle in real-time such as VMS (Variable Message Sign), Bus Lane Operation and LCS information.

3.2 Verification of each research technologies.

The verification will be evaluated based on performance objective presented institutes. Verification Methodologies will be created with reference to national and international standards.

Table 3. Integration of V2X communication test and service on the test bed
<table>
<thead>
<tr>
<th>Verification Technology</th>
<th>Verification Item</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chain Accident Prevention Service</td>
<td>· Information production rate : 10times/sec</td>
</tr>
<tr>
<td></td>
<td>· Accuracy : 100%</td>
</tr>
<tr>
<td></td>
<td>· Latency : under 0.1sec</td>
</tr>
<tr>
<td></td>
<td>· Accuracy of implementation : 100%</td>
</tr>
<tr>
<td>Approaching Emergency Vehicle Warning</td>
<td>· Information production rate : 10times/sec</td>
</tr>
<tr>
<td>Service</td>
<td>· Accuracy : 100%</td>
</tr>
<tr>
<td></td>
<td>· Latency : under 1sec</td>
</tr>
<tr>
<td></td>
<td>· Information production accuracy : 100%</td>
</tr>
<tr>
<td></td>
<td>· Accuracy of implementation : 100%</td>
</tr>
<tr>
<td>Emergency Call or SOS Service</td>
<td>· Information production rate : 10times/sec</td>
</tr>
<tr>
<td></td>
<td>· Accuracy : 100%</td>
</tr>
<tr>
<td></td>
<td>· Information production accuracy : 100%</td>
</tr>
<tr>
<td></td>
<td>· Accuracy of implementation : 100%</td>
</tr>
<tr>
<td>Self Vehicle Information Guide Service</td>
<td>· Information production rate : 10times/sec</td>
</tr>
<tr>
<td></td>
<td>· Accuracy : 100%</td>
</tr>
<tr>
<td></td>
<td>· Information collection rate : over 90%</td>
</tr>
<tr>
<td></td>
<td>· Information production accuracy : 100%</td>
</tr>
<tr>
<td></td>
<td>· Accuracy of implementation : 100%</td>
</tr>
<tr>
<td>Event Share Service</td>
<td>· Accuracy : 95%</td>
</tr>
<tr>
<td></td>
<td>· False alarm rate : under 10%</td>
</tr>
<tr>
<td></td>
<td>· Packet transmit rate : 100%</td>
</tr>
<tr>
<td></td>
<td>· Information expression accuracy : 100%</td>
</tr>
<tr>
<td></td>
<td>· Information production accuracy : 95%</td>
</tr>
<tr>
<td></td>
<td>· Accuracy of implementation : 100%</td>
</tr>
<tr>
<td>Construction Zone Warning Service</td>
<td>· Accuracy : 95%</td>
</tr>
<tr>
<td></td>
<td>· False alarm rate : under 10%</td>
</tr>
<tr>
<td></td>
<td>· Packet transmit rate : 100%</td>
</tr>
<tr>
<td></td>
<td>· Information expression accuracy : 100%</td>
</tr>
<tr>
<td></td>
<td>· Information production accuracy : 95%</td>
</tr>
<tr>
<td></td>
<td>· Accuracy of implementation : 100%</td>
</tr>
<tr>
<td>WAVE Tolling Service</td>
<td>· Accuracy : 99%</td>
</tr>
<tr>
<td>In-vehicle Signage</td>
<td>· Information production rate : 10times/sec</td>
</tr>
<tr>
<td></td>
<td>· Accuracy : 100%</td>
</tr>
<tr>
<td></td>
<td>· Latency : under 1sec</td>
</tr>
<tr>
<td></td>
<td>· Information production accuracy : 100%</td>
</tr>
<tr>
<td></td>
<td>· Accuracy of implementation : 100%</td>
</tr>
</tbody>
</table>

4. OPERATION RESULTS OF TEST BED

4.1 Analyzing V2X Communication Service Performance
A. Collecting event logs of all services and evaluating communication performance
   - All RSEs and OBEs collect logs for tx/rx events of all services
   - All logs are uploaded to LOG Server in SMART Highway Center
   - Logs are analyzed for evaluating communication performance
B. Collecting 21 types of logs
   - 2 for System stability and 19 for service performance

Table 4. Status & Service of Log Types

<table>
<thead>
<tr>
<th>Status &amp; Services</th>
<th>Log Types</th>
</tr>
</thead>
<tbody>
<tr>
<td>System Status</td>
<td>RSE / OBE</td>
</tr>
<tr>
<td>Connection info</td>
<td>RSE / OBE</td>
</tr>
<tr>
<td>Virtual VMS</td>
<td>RSE Tx / OBE Rx</td>
</tr>
</tbody>
</table>
4.2 Analyzing the operation result of SMART Highway test bed
A. Analyzing Object : WAVE RSE, SMART-I, Road Information Detection Radar
B. Collected Contents : System Halt, System Failure, Action
C. Expected Effect : Prior Action & Verification of Problem Recognized by the SMART Highway Test Bed in case of SMART Highway Commercialization

Table 5. Status of WAVE RSE

<table>
<thead>
<tr>
<th>RSE No</th>
<th>Reset of Power Supply</th>
<th>Network Disconnection</th>
<th>Total Working Time</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Event Date</td>
<td>Event Time</td>
<td>Event Date</td>
</tr>
<tr>
<td>1</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 6. Weekly Log Form of In-vehicle Signage

<table>
<thead>
<tr>
<th>No</th>
<th>Terminal No</th>
<th>Accepting(Terminal)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>RSE No</td>
<td>Rx count</td>
</tr>
<tr>
<td>1</td>
<td>B1</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>CB</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>...</td>
</tr>
</tbody>
</table>

Table 7. Weekly Log Form of Event Share Service

<table>
<thead>
<tr>
<th>Detection Date</th>
<th>No</th>
<th>Obstacle Longitude</th>
<th>Obstacle Latitude</th>
<th>Detection Time</th>
<th>RSE No</th>
<th>Start Time</th>
<th>Finish Time</th>
<th>Packet</th>
<th>Accepting(Terminal)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 8. Weekly Log Form of V2X Service(Construction Zone Warning Service)

<table>
<thead>
<tr>
<th>Date</th>
<th>Terminal</th>
<th>Event Time</th>
<th>Finish Time</th>
<th>Accepting(RSE)</th>
<th>Accepting(Terminal)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>RSE No</td>
<td>Rx count</td>
<td>Max. Distance</td>
<td>Min. Latency</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>000</td>
<td>1</td>
<td>8F</td>
<td>7</td>
<td>53</td>
<td>473</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>AF</td>
<td>3</td>
<td>36</td>
<td>925</td>
</tr>
</tbody>
</table>
First of all, we will write these weekly log form, then we will verify the verification items about each V2X services.

4.3 Analyzing the traffic safety improvement level
We will analyze the traffic safety improvement level based on V2X communication log data. We expect to induce the safe driving through the analysis of driving pattern.
   A. Confirmation of speed variance(acceleration, deceleration)
   B. Analysis of driver’s behavioral change
   C. Analysis of driver’s recognize time reduction
   D. Analysis of driver recognition response about incident information

5. CONCLUSION
In this study, we will check building status and utilization of test bed. Especially, we expect to improve the performance of project achievement and potential of spread. Finally, we will check the need of expansion of test bed from the large-scale demonstrations and a 24-hour operation.

6. ACKNOWLEDGMENT
This research was supported by a grant from Construction Technology Innovation Program(CTIP) funded by Ministry of Land, Transportation and Maritime Affairs(MLTM) of Korean government.

7 REFERENCES
[1] SMART Highway Research and Development Project, 2013, Korea Expressway Corporation
[3] The evaluation of WAVE communication under high speed environment for V2X services, ITS KOREA
<table>
<thead>
<tr>
<th>PAPER TITLE (90 Characters Max)</th>
<th>Research on Advanced Road Management using “ITS Spot” in Japan</th>
</tr>
</thead>
</table>

**TRACK**

<table>
<thead>
<tr>
<th>AUTHOR (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shoichi SUZUKI</td>
<td>Senior Researcher</td>
<td>ITS Division, National Institute for Land and Infrastructure Management, Ministry of Land, Infrastructure, Transport and Tourism</td>
<td>Japan</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CO-AUTHOR(S) (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Takahiro TSUKII</td>
<td>Researcher</td>
<td>ITS Division, National Institute for Land and Infrastructure Management, Ministry of Land, Infrastructure, Transport and Tourism</td>
<td>Japan</td>
</tr>
<tr>
<td>Hiroshi MAKINO</td>
<td>Head</td>
<td>ITS Division, National Institute for Land and Infrastructure Management, Ministry of Land, Infrastructure, Transport and Tourism</td>
<td>Japan</td>
</tr>
</tbody>
</table>

**E-MAIL (for correspondence)** suzuki-s92tg@nilim.go.jp

**KEYWORDS:** probe data, ITS Spot, evaluation, heavy vehicle monitoring

**ABSTRACT:**

In Japan it is required to implement road management under strict financial constraints and its decreasing population. Therefore a kind of Cooperative ITS called "ITS Spot Service" was launched nationwide in 2011 in order to make road transport safer and more efficient, and to improve road management. More than sixteen hundreds of road side unit (RSU) named "ITS Spot" were installed mainly along expressways and three Day-1 applications have been provided via communications between ITS Spots and ITS Spot compatible on-board units (ITS Spot OBUs), which more than three hundred thousand users bought and installed on their vehicles.

First this paper illustrates the outline of the "ITS Spot Service" especially on its day-1 applications and shows the result of evaluation, a series of user satisfaction survey on them. Secondly, some results of analysis are shared to indicates effectiveness of probe data which collected by ITS Spot for efficient road management. Finally, it reports an ongoing verification test in large scale on an application for heavy vehicle monitoring using probe data collected by ITS Spots, which is one of the Day-2 application candidates of ITS Spot services.
1 INTRODUCTION

As same as other countries in the world, road authorities and operators have been constructing and improving road infrastructure which leads to economic growth and better social welfare in Japan. They also have been tackling to reduce adverse effects caused by tremendously increasing road traffic such as time loss and environmental deterioration caused by traffic congestions and casualties by traffic accidents on the roads by various measures. ITS, Intelligent Transport Systems, is one of the such measures which employs information technologies to solve the problems above mentioned. Since late 1990's nationwide implementations of ITS services have started in Japan. Accompanying broad deployments of car navigation systems, VICS (Vehicle Information and Communication System) service has launched. And ETC (Electronic Toll Collection) service has also begun nationwide in early 2000's. VICS service provides real time road traffic information to drivers through car navigation systems’ display and helps drivers to avoid congested routes. ETC service enables drivers to pay their expressway toll via radio communication between ETC on-board unit and roadside antenna without stopping at toll gates. It has reduced traffic congestion at toll gates dramatically. Besides, ETC service provides advanced means to express operators implementing flexible and dynamic toll collection for travel demand management. For example, drivers who pass the Metropolitan Expressway during off-peak time are able to get toll reduction if they use ETC service for payment. That incentive provision program can shifts the traffic demand during peak time to off-peak. Both of two ITS services have been widely accepted by drivers in Japan and the numbers of users of each service are around 42 million and 41 million respectively as of end-March, 2014. As the first stage of ITS service deployment, VICS and ETC achieved great success with huge number of service users.

After such successful deployment of ITS services, the Smartway Project Advisory Committee (chairman: Shoichiro Toyoda, Honorary Chairman of Japan Federation of Economic Organizations at that time) published a proposal titled “ITS Enter the Second Stage: Smart Mobility for All” in August 2004 (Fujimoto et al. 2010). It is insisted that it is necessary to provide a variety of ITS services with a unified on-board unit (OBU) through vehicle-infrastructure communications in order to achieve safer, smoother, and more comfortable road traffic. It is also indicated that even technology development in ICT let various ITS services available, it would be inconvenient, unsafe and inefficient if each ITS service require OBU or roadside unit (RSU) respectively because they would occupy the limited space around the driver’s seat and attention, and require additional investments for each OBUs and RSUs every time when new service is launched.

Responding to the proposal, Ministry of Construction, the former body of Ministry of Land, Infrastructure, Transport and Tourism (MLIT), has urged realization of proposed concept and led the development of next-generation road services, which would enable the various ITS services available through a unified OBU in addition to ITS services which are already in operation and broadly deployed such as car navigation systems, ETC system and VICS. Based on research and development for years “ITS Spot Service” has launched as the next-generation road service in 2011 finally (Ueda et al. 2011). It utilizes dedicated short range communication (DSRC), conforming to the standards set of ISO 15628, between unified OBU, named “ITS Spot-compatible on-board unit” (ITS Spot OBU) and roadside unit named “ITS Spot” as shown in Figure 1.

Figure 1. Schematic diagram of ITS Spot communication and service example
According to the concept proposed by the Smartway Project Advisory Committee, ITS Spot service has the clear advantages as the followings. Firstly ITS Spot OBU's can communicate not only with ITS Spots but also with existing ETC antennas at expressway toll gates locating more than 1,500 places. Besides ETC service users can keep using their ETC payment accounts even after they change over from ETC OBU to ITS Spot OBU. Therefore ITS Spot service can provide the services which are regarded as a kind of value added services of ETC service, as shown in Figure 2. In other words, drivers who start using ITS Spot service can enjoy the same level of ETC service as an application of ITS Spot service. Secondly ITS Spot OBU can receive larger volume of real-time road traffic information than VICS service in virtue of DSRC with ITS Spots, and can utilize it for various mobility and safety applications. With sufficient ITS Spots installation, drivers can enjoy ITS Spot service from the day they put ITS Spot OBU on their vehicle no matter what penetration rate of OBUs is, while certain amount of deployment is essential to work for other kind of ITS service which rely on Vehicle-to-Vehicle communication. Thirdly ITS Spot OBU employs expanded message set of VICS service so that road operators can utilize legacy systems such as data base and communication interface among traffic control centers and institutional framework of road traffic information handling among road authorities and operators. Finally ITS Spot OBU can not only receive information from ITS Spot but also send information, called “ITS Spot probe data” which are generated by onboard sensors, without telecommunication cost thanks to DSRC.

![Figure 2. Integration and evolutions of ITS services in Japan](image)

In this paper, the outline of the Day-1 applications of ITS Spot service is described and the summary of satisfaction survey results on them is reported. Then, some examples of analysis using ITS Spot probe data, which road operators could possibly utilize in order to implement advanced road management are reported. Thirdly, it is reported that new R&D activities are on-going to use ITS Spot probe data with various information and data from other sources for heavy vehicle monitoring aiming more effective asset management of road infrastructure.

### 2 DAY-1 APPLICATIONS AND THE RESULTS OF SATISFACTION SURVEY

In 2011, ITS Spots were installed at more than 1,600 locations mainly on expressways throughout Japan (Ueda et al. 2011). And the ITS Spot service, the world-first operational Cooperative ITS service, was launched nationwide in August 2011. Drivers are able to enjoy ITS Spot services around every 10 to 15 kilometer including the points of before junction (around 90 points) on inter-urban expressways, and around every 4 kilometers on inner-urban expressways with their ITS Spot OBUs. Besides, around 50 ITS Spots are installed at rest area along expressway including Tomei and Meishin Expressway and Michi-no-Eki, rest areas along trunk roads. With regard to OBUs, more than 24 manufacturers deal and ship ITS Spot OBUs and more than 384 thousand ITS Spot OBUs have been installed on vehicles as of end-August, 2014.

Several applications of ITS Spot service are provided through DSRC between ITS OBU and ITS Spot. Figure 3 shows the applications of ITS Spot Service starting in 2011. Day-1 applications consist of three main applications for users, namely mobility applications, safety applications and ETC application. Mobility applications consist of several applications such as dynamic route guidance, route selection support information in wide area, travel time information to destinations and still image information of traffic condition. Figure 3(a) shows the examples of images on navigation...
display of mobility applications. Safety applications also consist of several applications such as congestion tail information, caution on accident prone spot, caution on road works and obstacles, weather information, still image information of road surface condition and emergency information. Figure 3(b) shows examples of safety applications. ETC application is fully compatible with existing facilities of the ETC service as above mentioned, which has been under operation since 2001 and its penetration rate at toll gates in the nation has reached around 90% averagely.

![Examples of image on car navigation screen of mobility applications](image1)

**Figure 3. Examples of mobility and safety applications**

In order to evaluate effectiveness and find needs for improvement of the day-one applications of ITS Spot service, MLIT has conducted satisfaction surveys throughout Japan (Kanazawa et al. 2012a). Table 1 shows the number of answering respondents and occasions of the surveys. To conduct a monitoring survey using an online questionnaire system, approximately 560 ITS Spot-compatible car navigation systems were distributed to “monitors”, i.e. individuals such as general drivers, logistics service drivers, bus drivers, taxi drivers, and rental car drivers. 94% of monitors have more than 10-year experience of driving, and 41% of them drive daily and 80% of them drive more than once a week. With regard to the monitors’ vehicle type, around 84% of them are passenger vehicles and the rest of them are commercial vehicles such as coaches and trucks. The degree of effectiveness of the services was surveyed, with the reasons and circumstances in which the services were evaluated as effective. The degree of effectiveness represents the ratio of people who experienced the services and evaluated it as “Very effective” and “Relatively effective”; that is, the ratio of monitors that appreciated the effects of the services.

**Table 1. Occasions and answering respondents of the survey**

<table>
<thead>
<tr>
<th>No.</th>
<th>Occasion of survey</th>
<th>Period after service commencement</th>
<th>Number of answering respondents</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Jan. 2012</td>
<td>5 month later</td>
<td>495</td>
</tr>
<tr>
<td>2</td>
<td>Nov. 2012</td>
<td>15 month later</td>
<td>467</td>
</tr>
<tr>
<td>3</td>
<td>Dec. 2013</td>
<td>28 month later</td>
<td>419</td>
</tr>
</tbody>
</table>

Figure 4 shows the result of the survey on major mobility applications. “Dynamic route guidance” was received favorably by around 63%, “Route selection support information in wide area” was received favorably by around 50%, “Travel time information to destinations” was received favorably by around 70% and “Still image information of traffic condition” was received favorably by around 35% in 2013. The reasons why the respondents evaluated the ITS Spot Services as effective were summarized as follows: “Being able to avoid congestions,” “Reduction in travel time” and “Sense of relief by being informed of the estimated arrival time.” Most drivers who evaluate “Dynamic route guidance” say it was more beneficial when they drove the routes on which they drive for the first time rather than on their ordinary route. Figure 5 shows the result of the surveys on major safety applications. “Congestion tail information” was received favorably by around 79%, “Caution on accident prone spot” was received favorably by around 69%, “Caution on road works and obstacles” was received favorably by around 80%, “Weather information” was received favorably by around 77%, “Still image information of road surface condition” was received favorably by around 58% and “Emergency information” was received favorable by 82% in 2013. The results indicate that the safety applications were valued over the mobility applications by users. The reasons why the monitors felt effectiveness were summarized as follows: “Being alerted of troubles ahead” and “Being able to slow down ahead of
time.” Some drivers stated that “Caution on accident prone spot” was found the most beneficial in case they drove unfamiliar routes.

Figure 4. Result of the survey on major mobility applications

![Image showing the result of the survey on major mobility applications]

Figure 5. Result of the survey on major safety applications

It is supposed that safety applications are evaluated more highly than mobility applications because only ITS Spot Service can provide valuable information or effective cautions to drivers for preparing against dangerous situation on the roads appropriately. For example, Figure 1 illustrates typical combination of facilities for the application of congestion tail information. Shown as figure 1, ITS Spot provides caution to drivers through ITS OBU, who are approaching congestion tail behind a blind curve. Because cautions are generated based on information detected roadside sensors, ITS Spot can provide them only when congestion occurs behind the curve with appropriate distance with which drivers can prepare and take action to avoid dangerous situation in advance. In short this application has advantage to autonomous type of ITS application which uses only various sensors onboard so that it is difficult to
inform drivers with adequate time in advance. The effectiveness of information provision using ITS Spot OBU had been verified in field operational tests held on Expressways. The results indicated that information provided through ITS Spot could be recognized by around 90% of drivers though information displayed on a Valuable Message Sign on the roadside could be recognized by around 50% (Kanazawa et al. 2010). Compared to safety applications, mobility applications might provide redundant information to drivers to some extent so that drivers feel ineffectiveness on them.

3 EXAMPLES OF ANALYSIS USING ITS SPOT PROBE DATA

As mentioned above it is possible that ITS Spot OBUs upload probe data which are generated by onboard sensors to ITS Spots. Figure 6 illustrates a flow of probe data from ITS Spot OBUs via ITS Spots and the Probe server to road administrators/operators. The ITS Spot OBU has the capacity of storing and uploading probe data accumulated around 80km-100km. Therefore probe data sent to the ITS Spots can include data recorded not only on expressways but also on ordinary roads, though most of ITS Spots are installed on expressways. The road network on which probe data have been collected as of April, 2014 are also indicated in Figure 6. The ITS Spot probe data is composed of three records, namely, “Basic information,” “Travel records” and “Behavior records” (Kanazawa et al. 2012b). The ITS Spot probe data collected through above mentioned system is designed for road traffic information generation. That is after wide deployment of ITS Spot OBUs, various vehicles’ information such as travel speed, route selection and hard braking in broad area would be obvious without additional investment for a number of monitoring camera, speed-counters and so on along roadside. And it would be possible for road operators and authorities to understand real traffic situations precisely and to implement advanced road management.

NILIM started development of a system which utilizes ITS Spot probe data for road traffic analysis a couple of years ago. After the analysis of the business processes of road administrators, NILIM identified some candidate applications which might be executed with the ITS Spot probe data, that is, “Travel speed survey,” “Congestion length survey,” “Routing study,” “Impact assessment of road works,” “Monitoring of unusual events on road network,” “Monitoring of trafficable route in case of disasters” and “Identifying near-accident prone spots.” Based on those candidates, NILIM developed a prototype system of “Probe information utilization system” considering necessary data volume, aggregation intervals, current data collection situation, etc. The prototype system and sample data was distributed among 9 regional development bureau offices of the MLIT and 6 expressway companies to be verified its performance and usability. NILIM conducted interviews and a questionnaire survey with whom the prototype system had been delivered in order to improve the system. In 2013 the probe information utilization system has begun its operation, with a multiple route selection function in addition to three applications of the prototype system.
Examples of analysis output by the probe information utilization system are shown in Figure 7. Average travel speed at arbitrary time of each road section can be displayed in colors as shown in Figure 7(a), so that road operators can scan the traffic situation of any area in order to find problematic points such as bottle necks. Users of the system can scrutinize specific point by mapping sudden decelerating point as shown in Figure 7(b) and by drawing a time-space diagram as shown in Figure 7(c). The former one might help users to find candidate spots for road safety measures as accumulated data on sudden decelerating points suggest accident prone spots. The latter one might provide precise information about the time when traffic congestions occur and the place the congestion queue starts and ends. Therefore road operators might be able to optimize timing and duration of road works and evaluate countermeasures against repetitive traffic congestions using the time-space diagram writer function.

As above mentioned the number of ITS Spot OBUs on the ground is around 400 thousand so that it is difficult that traffic information generated by ITS spot probe data fill a same role as by conventional facilities yet. However, it is expected that ITS spot probe data is going to contribute to more advanced road operation and management with broadly deployed ITS Spot OBUs in several years. Some road operators have already started to utilize ITS Spot probe data using the probe information utilization system only in limited area under the restriction of data amount and they anticipate that volume and distribution of data will increase up to sufficient level for practical use.

(a) travel speed mapping  
(b) sudden deceleration points mapping  
(c) time–space diagram writer  

Figure 7. Examples of analysis by the probe data utilization system

4 HEAVY VEHICLE MONITORING USING ITS SPOT PROBE DATA

As same as many countries in the world it has been one of the serious problems in Japan that over load heavy vehicles pass through and cause damage on road infrastructure of lower level of category so that designed life span of those infrastructure might be decreased and road authorities might been obliged to owe additional cost for maintenance and renewal of them. In Japan, it is required by a law that whoever planning to drive an over specification vehicle in terms of weight, size, shape and etc. apply and get a permission issued by road authority. The permissions regulate authorized routes, driving time, travel speed and so on for each application. Though, checking and monitoring activities
for law enforcement have been inadequately conducted at limited number of inspection places. From the view point of fleet companies the application procedure imposes some burden on them as the period of validity of permissions are less than two years and they have to repeat paper works. It was announced by MLIT in May 2014 that it tackles the heavy vehicle issues above mentioned with holistic approach and heavy vehicle monitoring with ITS Spot technologies would be one of the various measures. Indeed, MLIT has already started large scale field test with support of NILIM in order to verify the heavy vehicle monitoring service, which would be launched in a couple of years as a part of the day-2 applications of ITS Spot service. The concept of heavy vehicle monitoring service is shown in figure 8. Under the new over-specification-vehicle-permission program, logistic companies would be able to operate their trucks with permissions of extended validity if they equip ITS Spot OBU and register identification information to the authority. At the same instant it is confirmed that logistic company consents to provide individual ITS Spot probe data to the road authority for their use of heavy vehicle monitoring and of road management. Road authority cross-checks monitored route and weight collected by ITS Spots and weigh-in-motion (WIM) stations with permission condition using a condition check system. Extended permission would be cancelled if it is confirmed that a truck violates the conditions set by its permission.

![Figure 8. Concept of the heavy vehicle monitoring service](image)

It is planned that collected ITS Spot probe data via ITS Spots from more than 3,000 trucks joining the field test will be merged with weigh monitoring data at WIM stations on national highways (Suzuki et al. 2013). Then it will be cross-checked whether the merged traffic data match dairy operation reports made by logistic companies or not, in terms of origin, destination, stopping point, driving distance, driving time, kinds of selected route and etc. MLIT is planning to amend the institution of the over specific vehicle permission based on the results of verification test in a couple of years. It is expected that the launching of the heavy vehicle monitoring application as a part of day-2 applications of ITS Spot service will enhance more broader deployment of ITS Spot OBU and upgrade the level of road management with enormous probe data.

5 CONCLUSIONS

This paper illustrates the outline of "ITS Spot Service" especially on its Day-1 services, and the result of user satisfaction surveys on them. It is indicated that most of applications is accepted preferably by drivers and that safety applications are more highly evaluated than mobility applications. To reduce traffic accidents on the road is one of the most important challenges for road operators and society. ITS Spot service would be considered as one of the effective measures against it. It is necessary to verify quantitatively the impact of safety applications of ITS Spot service on drivers’ safety in general terms. Besides it is necessary to check and improve mobility applications narrowing down information not to be excess amount. Secondly, some outputs of analysis using probe data which collected by ITS Spot and processed by the probe data utilization system are reported. It is implied that road management and planning would be advanced with less cost by using probe data with which road operators can find sudden braking spots and traffic jam starting points as a candidate location where they should implement some measures. It is considered that increase of ITS Spot probe data would contribute to raising accuracy and versatility in terms of area of analysis. It is also necessary to improve the methods and the system of probe utilization based on actual usage by various road operators under real situations. Finally, it reports an ongoing verification test in large scale on the heavy vehicle monitoring application which is expected to be a part of day-2 applications of ITS Spot services. Traffic records generated from ITS Spot probe
data and weight records scaled at WIM stations are cross-checked with dairy operation reports in order to verify the accuracy and find things to keep in mind in order to amend the institutional framework, which is expected in a couple of years time.

REFERENCES


Kanazawa et al. (2010). PROVING TEST TO DEVELOP A PRACTICAL SERVICE TO OPTIMIZE LANE UTILIZATION RATES AT SAG SECTIONS OF EXPRESSWAYS, Proceedings of the 17th ITS World Congress, Busan.


Kanazawa et al. (2012b). APPLICABILITY OF A PROBE DATA COLLECTION SYSTEM TO ROAD TRAFFIC MANAGEMENT, Proceedings of the 19th ITS World Congress, Vienna.

Suzuki et al. (2013). A Verification Test on Heavy Vehicle’s Travel Speed Monitored by “ITS Spot”, Proceedings of the IRF 17th World Meeting & Exhibition, Riyadh.

### PAPER TITLE

<table>
<thead>
<tr>
<th>TRACK</th>
<th>Road Safety</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>AUTHOR (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jiyeon HONG</td>
<td>Research Professor</td>
<td>University of SEOUL</td>
<td>Korea</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CO-AUTHOR(S) (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sooboeom LEE</td>
<td>Professor</td>
<td>University of SEOUL</td>
<td>Korea</td>
</tr>
<tr>
<td>Joonbeom LIM</td>
<td>Ph. D Candidate</td>
<td>University of SEOUL</td>
<td>Korea</td>
</tr>
<tr>
<td>Jeonghyun KIM</td>
<td>Principal Researcher</td>
<td>Korea Railroad Research Institute</td>
<td>Korea</td>
</tr>
</tbody>
</table>

| E-MAIL (for correspondence) | cathy56@uos.ac.kr |

### KEYWORDS:

Traffic Accident Predication Model, Spatial Correlation, Spatial Econometrics, Moran’s I, Hot-spot Analysis

### ABSTRACT:

This study aims to discover information helpful in forming a safe urban environment by analyzing accident occurrence characteristics and influence factors of traffic accidents. For this purpose, LISA (Local Indicator of Spatial Association) analysis and hot spot analysis of traffic accidents were conducted using a GIS and spatial statistical analysis was performed.

The major results analyzed in this study are as follows. First, traffic accidents in each zone showed clustered characteristics which were spatially adjacent and there was a statistically marked correlation. Second, using the spatial statistic model, the factors related to traffic accidents were classified into accident exposure factors in a positive relationship with the accident occurrence frequency and traffic safety facility factors in a negative relationship with the accident occurrence frequency. Third, as accident exposure factors, road extension, the gross building floor area and the number of intersections and crosswalks were significant variables. Fourth, as traffic safety facility factors, the installation ratio of crosswalk pavement marking, the number of speed bumps and traffic violation control records by police force were significant variables.

These study results would serve as basic data for related policy to promote regional traffic accident prevention activities.
Analysis of Spatial Patterns and Influence Factors of Urban Traffic Accidents: A Case of Seoul, Korea
Ph.D Jiyeon HONG\textsuperscript{1}, Prof Soobeom LEE\textsuperscript{1}, Joonbeom Lim\textsuperscript{1}, Ph.D Jeonghyun KIM\textsuperscript{2}
\textsuperscript{1}The University of Seoul, Seoul, Korea
\textsuperscript{2}Korea Railroad Research Institute, Uiwang, Korea
Email for correspondence: cathy56@uos.ac.kr

1 INTRODUCTION

This study focuses on a phenomenon that all activity of humans always occurs with both elements of time and space. Traffic accident is the most representative phenomenon causing negative result among several results of human activity. It occurs at specific time and location, but includes the comprehensive interaction between spatial road environments, human factors and potential factors during the process. What's representative among such potential factors is spatial interaction (Shinhyung PARK, 2010). Here, spatial interaction means that spatial objects influence and are influenced each other at a geographic space.

Many researchers have recognized the influences of the geographical space, i.e. roads, and the efforts have been given to consider the influences of space to the traffic accidents. After the studies of the spatial analyses for the traffic accident data by Black(1992) and Loveday(1992), the spatial patterns of traffic accidents and the autocorrelations were reviewed by N. Levine(1995), G. Lee(2004) and Y. Lee(2007). N. Levine(1995) examines spatial patterns in motor vehicle crashes for the city. The additional studies were then followed by Benoît Flahaut(2004), Mohammed A. Quddus(2008), Cha Wang(2009), and S. Park(2010).

This study aims to find the influence factors of traffic accident to reflect such spatial interaction. Thus, It can then discover useful information for creating a safe urban environment.

2 THEORY AND METHODOLOGY

THEORY OF SPATIAL ANALYSIS

Econometric researches on various models that include two factors of time and space have been conducted continuously. If Time Series Analysis dealing with time directly and Cross Sectional Data include temporal factors, Spatial Econometrics or Spatial Statistics include spatial factors. Before defining a spatial econometric model, spatial dependence and spatial heterogeneity should be understood first.

LeSage(1995) and LEE et al.(2006) stated that spatial dependence is a property that contains a correlation between dependent variables. In other words, variable $y_i$ in region $i$ has the same relation as formula (1). This suggests that as a dependent variable in one region is identified by a dependent variable in another region, dependent variable serves as independent variable (LEE et al., 2006). Tobler(1970)'s first law of geography indicating that 'Everything is related to everything else, but near things are more related than distant things’ is explanatory of this spatial dependence. BYUN(2007) and Fotheringham et al.(2002) define spatial dependence as 'A property in one region being influenced by a property in other adjacent regions' as in formula(1).

$$ y_i = f(y_i), i = 1, 2, ..., n \quad i \neq j $$

(1)

Spatial heterogeneity means a difference in spatial characteristics that may be raised if a particular model is applied into other different regions (Anselin, 1992). In other words, spatial heterogeneity means that the impact of space on determination of dependent variables does not appear uniformly and thus variable $y_i$ in region $i$ has the same correlation as formula (2)(LEE et al., 2006).

$$ y_i = f_i(X_i\beta_i + \epsilon_i) $$

(2)

1 As all data are explanatory of the results occurred at a particular time, it can be said that researches using cross-section data contain temporal factors (LEE et al., 2006).
PARK(2010) suggested that spatial heterogeneity is caused by spatial or regional difference because spatial object has a unique different location. BYUN(2007) and Smith et al.(2008) indicated that a result analysed from a certain limited region can be different from the result obtained from another region. Like this, spatial dependence and spatial heterogeneity can't be reflected in general linear regression analysis.

Econometrics models containing spatial concept attempted various methods that may contain space as independent variable, as a means to solve this problem. As one of such methods, Spatial Weighted Matrix or Spatial Distance is suggested (Lee Sung-Woo et al., 2006).

Spatial Weighted Matrix is a quantitative expression of spatial relation between variables containing spatial characteristics. In other words, it is an expression in matrix of the relation between region $i$ and region $j$ within the analysed area. This Spatial Weighted Matrix is a very important concept. Test statistics of spatial dependence may be significant or not depending on the method or technique of calculating spatial weight (JUNG, 2011). Anselin(1992) asserted that the success or failure of the model depends on how to construct the model. Furthermore, many previous researches specify the importance of spatial weight (Cliff and Ord, 1981; Upton and Fingleton, 1985; Anselin, 1988).

Spatial weight is largely classified into contiguity-based weight granting values 0 or 1 and distance-based weight granting weight to the distance of actual analysed unit, depending on physical proximity (Contiguity) of analysed unit. Contiguity-based weight indicates 1 if adjacent and 0 if separated, considering spatial proximity and is the most fundamental weight of spatial econometrics and also called as Spatial Contiguity Matrix. LeSage(1999) classified contiguity into four sub-categories as in Figure 1 and proposed the following methods of granting weight.

![Spatial Weighted Matrix Diagram](image)

**Figure 1** Quantification of spatial proximity (Lesage, 1999)

This study hypothesized, like JIN et al.(2012), that as the spatial unit of this study, administrative spatial unit has various sizes and patterns, it is desirable to set Contiguity-based weight matrix rather than distance-based weight matrix. Of the Contiguity-based methods, Rook-Contiguity, a method to grant weight to the surface sharing lines is applied and ArcMap 10.0 Spatial Statistics Tools are used.

**VERIFICATION OF SPATIAL CORRELATION**

Spatial correlation may appear globally along the physical range of analyzed region or in some regions locally (JUNG, 2011). Fotheringham(2002) suggested that if counting the data of two different regions into one data, so-called Simpson's paradox(as following Figure 2) may appear, according to the logic of mean. For example, although two regions had a negative correlation between density of dwelling and price of land, a paradox that there exists a positive correlation if aggregating the two regions, may appear. This suggests that there might be various groups showing a localized correlation, but such groups sometimes wouldn't be measured with global correlation indicator.

![Spatial Correlation Diagram](image)

**Figure 2** Simpson's paradox in spatial data (Fotheringham, 2002)
Global spatial autocorrelation refers to the situation that particular variables have interconnection depending on spatial distribution across the entire analyzed area. The representative statistics that are used to test global correlation of spatial variables are Moran’s I (Moran, 1950), Geary’s C (Geary, 1954), etc and among which, Moran’s I is the most widely-used one.

Moran’s I statistic is a measurement scale to identify spatial autocorrelation and thus indicates the degree that the values of neighboring spatial units are spatially distributed in a similar way. Moran’s I compares the values of spatial units to calculate coefficient and if similar zones are distributed adjacent, the values of spatial units are calculated closely to 1 in terms of Moran’s I value, and if having random patterns, it’s close to 0. In addition, if calculated close to −1, it's interpreted as having a negative spatial correlation. This suggests that different values are distributed in adjacent locations spatially. Figure 3 is shown Moran’s I statistic and concept of spatial distribution.

![Figure 3 Moran's I statistic and Concept of spatial distribution (KIM, 2011)](image)

In Moran's I, the statistical test is carried out by Z-test and the test statistic is same as formula (3)

Moran’s I = \[ \frac{N \sum_{i,j} \omega_{ij} (X_i - \bar{X})(X_j - \bar{X})}{\sum_{i} \omega_{ii} (X_i - \bar{X})^2} \]  \hspace{1cm} (3)

where, \( X_i, X_j \) : values of space \( i \) and space \( j \), \( \bar{X} \) : mean of \( X \)
\( \omega_{ij} \) : spatial weights matrix, \( N \) : number of space

Herein, \( \omega_{ij} \) refers to Spatial Contiguity Matrix as mentioned earlier, and the sum of the weighted values for the spatial units related a unit was standardized by row (\( \omega_{ij} \)) and the value of “1” is given whereas the own weighted value is “0”. \( \omega_{ij} \) is the standardized spatial weights matrix and is same as formula (4)

\[ \omega_{ij} = \frac{\omega_{ij}}{\sum_{i,j} \omega_{ij}}, \omega_{ii} = 0 \]  \hspace{1cm} (4)

Anselin(1995) modifies Moran’s I and presents LISA(Local Indicator of Spatial Association), an indicator to measure local spatial dependency. This indicator is also called as Local Moran’s I and Local Moran’s I in region \( i \) is defined as formula (5). This statistics represents a positive (+) value if the weighted mean of the values in surrounding regions adjacent to the values of a particular region is similar, conversely, a negative (-) value if the values of weighted mean are different. LISA is an analysis to identify how spatial cluster patterns occur locally (JEONG et al., 2009).

Local Moran’s I = \( (X_i - \bar{X}) \sum_j \omega_{ij} (X_j - \bar{X}) \)  \hspace{1cm} (5)

Another localized spatial dependence statistics is Local \( G^*_i \) presented by Getis and Ord (1995). This statistics is a sort of spatial cluster analysis indicating that unlike local Moran’s I, it has a positive (+) value if variables in the surrounding region are higher than then mean, but a negative (-) value in the opposite case. So it's also called as Hot Spot Analysis. \( G^*_i \) can be calculated by the formula (6) and \( G^*_i \) does not require additional standardization process because the value itself is Z-value.

Getis – Ord \( G^*_i \) = \[ \frac{\sum_i \omega_{ij} (X_i - \bar{X}) \sum_j \omega_{ij} (X_j - \bar{X})}{\sum_i \omega_{ij}^2 - (\sum_j \omega_{ij})^2} \]  \hspace{1cm} \hspace{1cm}  \( S = \sqrt{\frac{\sum_i (X_i^2)}{N} - (\bar{X})^2} \)  \hspace{1cm} (6)
SPATIAL ECONOMETRICS ANALYSIS

Two kinds of the spatial econometrics models are applied to develop the traffic accident frequency prediction model in urban area such as the Spatial Autoregressive Model (SAR) and the Spatial Errors Model (SEM) both of which are a kind of global spatial regression models. The SAR includes the spatial correlation in the dependent variable as shown in the equation (7), and the SEM does the spatial correlation in the error term as in the equation (8). Each model was developed by the “Maximum Likelihood Estimation” with the “OpenGeoDa 1.2.0”.

\[ y = \rho W_y + X\beta + \epsilon \]  \hspace{1cm} (7)
\[ y = X\beta + \mu, \mu = \lambda W\mu + \epsilon \]  \hspace{1cm} (8)

where, \( y \): A dependent variable
\( W_y \): A spatially lagged dependent variable for spatial weights matrix \( W \)
\( \rho \) : The scalar for spatial lag coefficient
\( \beta \) : The parameters to be estimated
\( X \) : The matrix of exogenous explanatory variables
\( \mu \) : The error term expressing spatial dependence
\( \lambda \) : The spatial autoregressive coefficient

3 DATA DESCRIPTION

The input data were acquired from the police department (traffic accident statistics, number of police enforcements), the city statics (numbers of residents and households), the Seoul GIS Potal (numbers of crosswalks, speed humps and crosswalk acoustic signs, and statistics on exclusive bus lanes), Center for Seoul Metropolitan Transportation (OD by modes), and the Road Name and Address Management System (building floor area by use) as following Figure 5. The scope of this study was limited in City of Seoul in 2010 and the spatial analysis unit was based on the administrative units as following Figure 4. It is because all the data is collected by the administrative unit each year, and the data in 2010 was the most recent available. The data were collected by the administrative unit and the dataset based on the GIS.

Figure 4 Administrative Unit in City of Seoul

Figure 5 Data Description

<table>
<thead>
<tr>
<th>Road &amp; Land Use</th>
<th>Socio-Economic Factors</th>
<th>Traffic Safety Facilities &amp; Policy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Length(km)</td>
<td># of Residents</td>
<td># of Crosswalks</td>
</tr>
<tr>
<td># of Intersections</td>
<td># of Employees among Residents</td>
<td>Ratio of Advance Crosswalk Warning Signs(%)</td>
</tr>
<tr>
<td>Intersection Density(#/km)</td>
<td># of Employees in Area</td>
<td>Ratio of Crosswalk Acoustic Signals(%)</td>
</tr>
<tr>
<td>Total Length of Exclusive Bus Lane(m)</td>
<td>% of Registered Cars</td>
<td>Ratio of Remaining Time Signs on Crosswalks(%)</td>
</tr>
<tr>
<td>Ratio of Exclusive Bus Lane(%)</td>
<td>Traffic Volume(trip)</td>
<td># of School Zones</td>
</tr>
<tr>
<td>Total Length of One-way(m)</td>
<td></td>
<td># of Speed Humps</td>
</tr>
<tr>
<td>Ratio of One-way(%)</td>
<td></td>
<td># of Cat’s Eye Systems</td>
</tr>
<tr>
<td>Area(in)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
4 RESULTS

SPATIAL CHARACTERISTICS OF TRAFFIC ACCIDENTS BY ADMINISTRATIVE UNIT

To verify if there is a spatial correlation in traffic accidents, Moran’s I statistic of spatial autocorrelation, which is described in 'methodology', LISA(Local indicators of spatial association), and Hot-Spot Analysis(Getis-Ord $G^*_i$) are performed. As a result of analyzing the spatial patterns of traffic accident through Moran’s I, it was found that Moran’s I values were positive and spatial objects had similar values each other and tended to be clustered as following Figure 6.

![Figure 6 Moran's I of Traffic Accident Frequency](image)

Moran’s I is a global indicator that shows the autocorrelation of the entire research region with one value. But, if the research area is relatively large, within the area, autocorrelation may differ, depending on location. Therefore, this study looks at local spatial correlation through Anselin(1995)'s LISA(Local indicators of spatial association) analysis and Getis and Ord(1995)'s Local $G^*_i$, also called as Hot Spot Analysis.

After LISA analysis, four different types are drawn as following Figure 7: HH(High-High) type indicating that autocorrelation between the applicable region and the surrounding region has all high values, LL(Low-Low) type indicating that the applicable region and the surrounding region has all low values, LH(Low-High) type indicating that the statistics of the applicable

![Figure 7 LISA Cluster Map of Traffic Accident Frequency](image)
region is low, but the surrounding value is high, and HL(High-Low) type indicating that the statistics of the applicable region is high, but the surrounding values are low.

After Hot-Spot Analysis (Getis-Ord $G'_i$), this study analyzes a spatial cluster with a positive (+) value if the variable values in the surrounding area are higher than the mean of autocorrelation in traffic accidents and a spatial cluster with a negative (-) value in the opposite case and then draws the results into the map. In the figure 8, the portion indicated in red represents the hot spots that have higher values than the mean and the portion indicated in blue represents the cold spots that have lower values than the mean. It is found that such a portion is clustered.

![Figure 8 Hot-spot Analysis of Traffic Accident Frequency](image)

ESTIMATION OF URBAN TRAFFIC ACCIDENT PREDICTION MODEL THROUGH SPATIAL ECONOMETRICS

Estimation of urban traffic accident prediction model through spatial econometrics is taled as follows. According to the results from the SAR model, the elements of the spatial lag, $\rho$ was 0.120, and the t-statistic was 4.2306 which could be significant at 1% level of significance. The spatial correlation was also verified with the likelihood ratio of 16.64279 ($p=0.00005$). The result from the SEM model represented the $\lambda$ value of 0.2659 and the t-statistic of 4.044, which could be regarded to be significant at the significance level of 1%. The likelihood ratio was 13.3156 ($p=0.00026$), so the spatial correlation was verified.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Spatial Econometrics Models</th>
<th>SEM Model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SAR Model</td>
<td>SEM Model</td>
</tr>
<tr>
<td></td>
<td>coefficient of regression</td>
<td>t-statistic</td>
</tr>
<tr>
<td>Constant</td>
<td>2.958</td>
<td>19.619</td>
</tr>
<tr>
<td>Total Length of Roads</td>
<td>0.016</td>
<td>6.200</td>
</tr>
<tr>
<td>Ratio of Exclusive Bus Lane (%)</td>
<td>4.819</td>
<td>8.214</td>
</tr>
<tr>
<td>Total Building Floor Area $(m^2)$</td>
<td>1.4E-7</td>
<td>4.273</td>
</tr>
<tr>
<td># of Intersections (#)</td>
<td>0.030</td>
<td>8.689</td>
</tr>
<tr>
<td># of Crosswalks (#)</td>
<td>6.9E-5</td>
<td>2.828</td>
</tr>
<tr>
<td>Ratio of Advance Crosswalk Warning Signs (%)</td>
<td>-0.411</td>
<td>-1.901</td>
</tr>
<tr>
<td># of Speed Humps (#)</td>
<td>-0.001</td>
<td>-3.714</td>
</tr>
<tr>
<td># of Police Enforcements</td>
<td>-0.150</td>
<td>-3.089</td>
</tr>
<tr>
<td>$\rho$</td>
<td>0.120</td>
<td>4.230</td>
</tr>
<tr>
<td>$\lambda$</td>
<td>0.266</td>
<td>4.044</td>
</tr>
<tr>
<td>$R^2$</td>
<td>0.590</td>
<td>0.591</td>
</tr>
</tbody>
</table>

The likelihood ratio

1) Spatial Autoregressive Correlation Coefficient in SAR
2) Spatial Error Coefficient in SEM
32 factors were reviewed for the explanatory variables such as 9 socio-economic, 9 road and land use and 14 traffic safety facilities and policy ones. By the analyses of the correlations and the multicollinearity between the variables, five variables on road and land use and three on traffic safety facility and policy were selected. The variables on road and land use were the total length of roads, the total building floor area, the ratio of exclusive bus lane, and the numbers of intersections and crosswalks. The selected variables on traffic safety facility and policy were the ratio of advance crosswalk warning signs and the numbers of speed humps and the police enforcements.

The variables on road and land use are the exogenous variables which cannot be controlled for the traffic safety, even though those influence the traffic accident exposure directly related to the accidents positively(+). The variables on traffic safety facility and policy have the negative(-) relationship with the accident frequency, and can be controlled in terms of traffic safety improvement.

5 CONCLUSIONS

This study used a geographic information system for traffic accident density analysis, LISA analysis, and hot spot analysis to map the local traffic accident patterns and also spatial regression model for spatial statistical analysis to identify the relation between traffic accident frequency and influence factors.

In this study, the following was found traffic accidents in each zone showed clustered characteristics which were spatially adjacent and there was a statistically marked correlation. Using the spatial statistic model, the factors related to traffic accidents were classified into accident exposure factors in a positive relationship with the accident occurrence frequency and traffic safety facility factors in a negative relationship with the accident occurrence frequency.

As accident exposure factors, road extension, the gross building floor area, the number of intersections and crosswalks were significant variables. As traffic safety facility factors, the installation ratio of crosswalk pavement marking, the number of speed humps and traffic violation control records by police force were significant variables.

These study results would serve as basic data for related policy to promote regional traffic accident prevention activities and safety from traffic accidents. Moreover, there is a need to conduct more clear and detailed research on the relationship between traffic accident occurrence and traffic safety facility(policy) to prevent traffic accidents in future.

ACKNOWLEDGEMENTS

This research was supported by Basic Science Research Program through the National Research Foundation of Korea (NRF) funded by the Ministry of Education, Science and Technology (2012R1A1A2041296).

REFERENCES

Anselin, L. (1988), Lagrange Multiplier Test Diagnostics for Spatial
Lee, S.W., Yun, S.D., Park, J.Y., Min, S.H(2006), The Practice On Spatial Econometrics Models, Pakyoung Press, Seoul
Upton, G.J.G., Fingleton, B.(1985), Spatial Data Analysis by Example: Categorical and Directional Data. Wiley, Chichester, UK.
<table>
<thead>
<tr>
<th>PAPER TITLE</th>
<th>STUDY ON BETTER UTILIZATION OF NATURAL ROCK ASPHALT (ASBUTON) BETWEEN INDONESIA AND JAPAN</th>
</tr>
</thead>
<tbody>
<tr>
<td>TRACK</td>
<td></td>
</tr>
<tr>
<td>AUTHOR</td>
<td>POSITION</td>
</tr>
<tr>
<td>Kazunari HIRAKAWA</td>
<td>Researcher Specialist</td>
</tr>
<tr>
<td>CO-AUTHOR(S)</td>
<td>POSITION</td>
</tr>
<tr>
<td>Iwao SASAKI</td>
<td>Senior Researcher</td>
</tr>
<tr>
<td>Kazuyuki KUBO</td>
<td>Chief Researcher</td>
</tr>
<tr>
<td>Atsushi KAWAKAMI</td>
<td>Senior Researcher</td>
</tr>
<tr>
<td>Akira MARUYAMA</td>
<td>Supervisory Researcher</td>
</tr>
<tr>
<td>Madi HERMADI</td>
<td>Researcher</td>
</tr>
<tr>
<td>E-MAIL</td>
<td><a href="mailto:k-hirakawa55@pwri.go.jp">k-hirakawa55@pwri.go.jp</a></td>
</tr>
</tbody>
</table>

KEYWORDS:
AsButon, Guss asphalt mixture, Pavement, Trinidad Lake Asphalt, Full extracted asphalt

ABSTRACT:
The natural rock asphalt yield at Buton-island in Indonesia, known as AsButon, is expected to be utilized as natural resource for pavement works because of its huge deposit. Since 2011, Institute of Road Engineering (IRE, Indonesia) and Public Works Research Institute (PWRI, Japan) have been conducting the cooperation research program to seek for better utilization of AsButon. In this program, joint workshops have held in every several months and exchanged information and discussed about AsButon.

By for now, the physical and chemical properties of asphalt extracted from AsButon (pure-AsButon) had been analyzed in this research, and it was clear that pure-AsButon has a close tendency of Trinidad Lake asphalt (TLA). Generally, in Japan, TLA is applied to guss asphalt for waterproof of steel deck plate. Then laboratory tests were done about guss asphalt mixture using AsButon alternative for TLA (AsButon-guss). The results of these tests about AsButon-guss were met the Japanese standards of guss asphalt mixture. Therefore AsButon is regarded to be possible alternative for TLA.

This paper shows the achievement of cooperation research about AsButon until today. It is suggested that the necessity of technical standard and guideline to make AsButon better utilization for the future.
Study on better utilization of natural rock asphalt (AsButon) between Indonesia and Japan

Kazunari HIRAKAWA¹, Iwao SASAKI¹, Kazuyuki KUBO¹,
Atsushi KAWAKAMI¹, Akira MARUYAMA², and Madi HERMADI³

¹Public Works Research Institute, Tsukuba, Ibaraki, Japan
²Nichireiki Co., Ltd, Japan
³Institute on Road Engineering, Indonesia
Email for correspondence: k-hirakawa55@pwri.go.jp

1. INTRODUCTION

The natural rock asphalt yield at Buton-island in Indonesia, known as AsButon, is expected to be utilized as natural resource for pavement works because of its huge deposit. Since 2011, Institute of Road Engineering (IRE, Indonesia) and Public Works Research Institute (PWRI, Japan) have been conducting the cooperation research program to seek for better utilization of AsButon. In this program, joint workshops have held in every several months and exchanged information and discussed about AsButon.

By for now, the physical and chemical properties of asphalt extracted from AsButon (pure-AsButon) had been analyzed in this research, and it was clear that pure-AsButon has a close tendency of Trinidad Lake asphalt (TLA). Generally, in Japan, TLA is applied to guss asphalt for waterproof of steel deck plate. Then laboratory tests were done about guss asphalt mixture using AsButon alternative for TLA (AsButon-guss). The results of these tests about AsButon-guss were met the Japanese standards of guss asphalt mixture. Therefore AsButon is regarded to be possible alternative for TLA.

This paper shows the achievement of cooperation research about AsButon until today. It is suggested that the necessity of technical standard and guideline to make AsButon better utilization for the future.

2. ASBUTON

AsButon is a kind of rock asphalt deposits existing on Buton Island in Indonesia. The location of Buton Island is shown in Figure 1. AsButon reserves are estimated at 163,900,000 tons. The asphalt content of AsButon is 35% or less.

Table 1 shows estimated reserves of AsButon. In Indonesia, studies have been conducted on the use of AsButon in the macadam method for the construction of low-cost local roads, blending with petroleum asphalt and 100% extraction of asphalt.

<table>
<thead>
<tr>
<th>Region</th>
<th>Approximate Deposit (ton)</th>
<th>Bitumen Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Waisiu</td>
<td>100,000</td>
<td>~35</td>
</tr>
<tr>
<td>Kabungka</td>
<td>60,000,000</td>
<td>15-35</td>
</tr>
<tr>
<td>Winto</td>
<td>3,200,000</td>
<td>25-35</td>
</tr>
<tr>
<td>Wanit</td>
<td>600,000</td>
<td>~30</td>
</tr>
<tr>
<td>Lawele</td>
<td>100,000,000</td>
<td>15-30</td>
</tr>
<tr>
<td>Total</td>
<td>163,900,000</td>
<td></td>
</tr>
</tbody>
</table>
3. OUTLINE OF COOPERATION RESEARCH

3.1 Back ground

In cooperation with the IRE of Indonesia, PWRI has been conducting a study on methods for utilizing natural asphalt produced in Indonesia. The establishment of a method for utilizing natural asphalt is not only an urgent task for Indonesia but also an important research challenge for Japan, which is a resource-poor country. A research agreement was signed in 2009, and a research road map was made in 2011. A total of eight workshops (including all areas of research) have been held to date.

2009.06 Pre-meeting in Jakarta and Bandung
2009.11 Tsukuba, MoU(C) agreed in general
2010.03 Bandung at IRE (1st WS for targeting the themes)
2010.06 Tsukuba, joint meeting for Bali coming symposium.
2010.10 International Symposium in Bali (2nd WS to declare the strategy for cooperative researches)
2011.01 Jakarta (3rd WS to make the roadmaps)
2011.09 Study Tour in Tsukuba and Kansai
2011.06 Sulawesi (4th WS to make the roadmaps and site observation)
2011.09 Technological Tour in Japan for modernizing experimental equipment and facilities of IRE
2011.10 Lombok (5th WS/ Bilateral Seminar)
2012.03 Jakarta and Bandung (6th WS including River and Water Resource issue)
2012.06 Triilateral international interim report workshop
2013.01 Jakarta (joint meeting about AsButon research)
2013.08 Bandung at IRE (7th WS to prolong cooperation research)
2013.11 Tokyo (8th WS to discuss about AsButon Technical Center)
2014.03 Tsukuba (joint meeting and Study Tour)
3.2 Road map
The road map is as following
2011 Provision of AsButon samples in Indonesia
2012 Technical advice from Japan (PWR1) and laboratory tests (Japan Modified Asphalt Association)
2013 Development of AsButon utilization guidelines
2014-2016 New contents were added to road map
  Technical support to estimate AsButon center in Indonesia
  Research chemical characteristics of Pure-AsButon
  Utilization as alternative to petroleum asphalt

Figure 2. Visit to Buton Island (2011.06)  Figure 3. Study tour in Tsukuba (2014.03)

3.3 Out Puts
In this cooperation research, three utilization methods were proposed according to each treatment levels as follows.
Low-cost use: Utilization granular AsButon for macadam surface treatment or subbase stabilization, etc.
Normal use : Quality controlled application as an additive for mixture products.
Advanced use : Utilization Pure-AsButon (full extracted binder) as alternative petroleum asphalt.
The achievements of cooperation research about AsButon until today are described following sections.

Figure 4. The proposal of utilization AsButon
4. GENERAL PROPERTIES OF ASBUTON

In 2011, IRE provided two types of AsButon mineral sample to PWRI (KABUNGKA and LAWELE which is abundant AsButon). A series of evaluation tests were conducted with both AsButon and 60/80 grade asphalt (for comparison). Asphalt was extracted from AsButon according to JPI-5S-31-1998.

Physical properties were evaluated by standard binder tests (penetration, softening point, ductility, viscosity, flash point, and density). Chemical properties were evaluated by measuring asphalt composition and molecular weight with chemical analysis instruments. The results are shown in Table 2, Figure 5, and Figure 6.

At point of physical properties, both Extracted asphalts from AsButon (Pure-AsButon) are harder than 60/80 grade asphalt at normal temperature and more viscous at high temperature. Density is also heavier than conventional asphalt. On the other hand, comparing two types of AsButon, property of LAWELE is little similar to straight asphalt.

<table>
<thead>
<tr>
<th>Testing Types</th>
<th>KABUNGKA</th>
<th>LAWELE</th>
<th>StAs60/80</th>
<th>TLA</th>
</tr>
</thead>
<tbody>
<tr>
<td>PEN 1/10mm</td>
<td>3</td>
<td>41</td>
<td>67</td>
<td>2</td>
</tr>
<tr>
<td>S.P. °C</td>
<td>84.0</td>
<td>55.5</td>
<td>48.0</td>
<td>96.5</td>
</tr>
<tr>
<td>15°C Duct</td>
<td>0</td>
<td>33</td>
<td>150+</td>
<td>0</td>
</tr>
<tr>
<td>60°C Vis</td>
<td>113,000</td>
<td>1,030</td>
<td>205</td>
<td>-</td>
</tr>
<tr>
<td>Flash Point</td>
<td>Unmeasurable</td>
<td>206</td>
<td>334</td>
<td>254</td>
</tr>
<tr>
<td>Density g/cm³</td>
<td>1.109</td>
<td>1.063</td>
<td>1.039</td>
<td>1.405</td>
</tr>
</tbody>
</table>

Figure 5 shows results of asphalt composition analysis. Other natural asphalts (gilsonite and TLA) are arranged as comparison. From this figure, you can see both AsButons have similar composition. And this composition is close to TLA rather than straight asphalt. Gilsonite indicates quite a different composition from others.
The analysis results of molecular weight distribution are shown in Figure 6. AsButon shows high molecular weight contents than straight asphalt therefore “Mw” is higher. KABUNGKA has more high molecular weight contents and less low weight molecular contents than LAWELE.

![Figure 6. Molecular weight distribution](image)

5. LOW-COST USE

To utilize granular AsButon as road pavement material, PWRI and IRE discussed about AsButon technical manual, named as” The Implementation of Macadam AsButon Penetration Coating (LPMA-AsButon)” published in 2010. In this discussion, PWRI indicated some suggestions comparing with Japanese technical guideline.

PWRI’s suggestions are following.
1) Coverage for LPMA-AsButon, should consider using as sub base material.
2) Requirements for liquid asphalt and/or emulsified asphalt materials are should be mentioned.
3) Requirement (quality control) and measurement method should be mentioned.

On the other hand PWRI have proposed for future utilization as following.
1) Raw mineral and binder material application as a stabilization base material.
2) In place of the emulsion, spray it as tack or prime coat (alternative to emulsified asphalt).

![Figure 7. Macadam AsButon Penetration Coating](image)
6. Normal USE

As mentioned above, extracted asphalt from AsButon is heavy asphalt. Especially KABUNGKA is much harder than 60/80 grade asphalt. And AsButon has similar composition to TLA. Usually, TLA is employed as a binder of guss asphalt mixture in Japan. This guss asphalt mixture is applied for the base of steel plate deck pavements as water-proof layer. In following, the result of evaluation tests with AsButon as a material of the guss asphalt mixture.

6.1 Test Items

To evaluate potential of AsButon, the five items shown in Table 3, which are provided in the bridge deck pavement standard for Honshu–Shikoku bridge (HSBE standard), the waterproofing handbook for highway bridges, pavement investigation and a testing methodology handbook, were tested.

Table 4 shows the binder of guss asphalt. In this study, three types of blended ratio (by weight) of natural asphalt and petroleum asphalt was applied (15%:85%, 20%:80%, and 25%:75%). On the other hand, the binder content of guss asphalt mixture was kept constant.

<table>
<thead>
<tr>
<th>Item</th>
<th>Test method</th>
<th>Test temperature</th>
<th>Standard-specified Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Workability</td>
<td>Lauel fluidity test</td>
<td>240°C</td>
<td>20s or less</td>
</tr>
<tr>
<td>Dynamic stability</td>
<td>Wheel tracking test</td>
<td>60°C</td>
<td>300 times/min or more</td>
</tr>
<tr>
<td>Flexibility at low temp.</td>
<td>Bending test (Breaking strain, 50 mm/min)</td>
<td>-10°C</td>
<td>8.0 × 10³ or more</td>
</tr>
<tr>
<td>Adhesiveness</td>
<td>Tensile adhesive strength test</td>
<td>23°C</td>
<td>0.6 N/mm² or more</td>
</tr>
<tr>
<td></td>
<td>(Steel deck, mixture)</td>
<td>-10°C</td>
<td>1.2 N/mm² or more</td>
</tr>
<tr>
<td>Waterproof property</td>
<td>Permeability test under pressured water</td>
<td>Temperature correction</td>
<td>No leakage</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Mixed ratio of natural asphalt and petroleum asphalt(%)</th>
<th>Total Binder Content(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Natural asphalt</td>
<td>Straight-run asphalt(20/40)</td>
</tr>
<tr>
<td>AsButon KABUNGKA</td>
<td>15</td>
</tr>
<tr>
<td>AsButon LAWELE</td>
<td>20</td>
</tr>
<tr>
<td>AsButon LAWELE</td>
<td>25</td>
</tr>
<tr>
<td>TLA</td>
<td>25</td>
</tr>
</tbody>
</table>

6.2 Test Result

1) Workability

Lauel fluidity test result is shown in Figure 8. In the case of AsButon ratio of 25%, Lauel fluidity is over 20s. To meet standard specified value, AsButon can employ up to 21~22%. This indicates that fluidity comparable to or higher than that obtainable at the TLA ratio of 25% can be obtained under the same conditions.

Figure 8. Workability
2) Dynamic stability
Wheel tracking test result is shown in Figure 9. To meet standard specified value, it is enough about 17% of AsButon additive ratio. If the AsButon ratio is 25%, dynamic stability of AsButon can be obtained higher than that obtainable at the TLA ratio of 25%.

![Figure 9. Dynamic stability](image)

3) Flexibility at low temperature
Figure 10 shows the bending test results using small specimen of each mixture which indicate tendency of brittleness temperature. The inflection temperatures of bending strength of all AsButon were higher or same as that of TLA, therefore the AsButon have good flexibility at lower temperature than TLA.

![Figure 10. Flexibility at low temperatur(-10°C)](image)

4) Adhesiveness and Waterproof property
In tensile adhesive strength test (at 23°C and -10°C), and permeability test under pressured water test, all of cases meet standard specified value. Therefore, adhesion and waterproof property between the steel plate deck and the guss asphalt mixture using AsButon is considered to be quite good.

As fiscal 2012, there are 680,728 bridges over 2 m long in Japan. Of these, 158,897 bridges are over 15 m long, of which 60,432 (38%) with a total length of 4,722 km are steel bridges.

Mentioned as Table 3, there are six requirements for the guss asphalt mixtures in Japan. It is as workability, dynamic stability, flexibility at low temperature, adhesiveness (at 23°C and -10°C), and waterproof property. If AsButon ratio was 20%, the guss asphalt mixture could meet these six requirements. It suggested that amounts of AsButon could be available both Indonesia and Japan as alternative to TLA.
7. ADVANCED USE

To study utilization pure-AsButon as alternative to straight asphalt, a series of aging tests were conducted in laboratory. Figure 11 shows penetration and softening point under original condition and aged condition (after thin film oven test (TFOT) and Pressure Aging Vessel (PAV)). AsButon seems to be less affective to aging. However it is caused by original value is worse than straight asphalt. Therefore, In advanced use, two approaches are proposed. One is to make soften with oil. The other is usage as “a modifier” to improve petroleum asphalt.

![Figure 11. Penetration and softening point after aging in laboratory](image)

8. CONCLUSIONS

In this cooperation research program between Indonesia and Japan, method of utilization AsButon was found by combining materials and Indonesia and Japanese technology. In Normal use, it is clear that AsButon can be used as a binder of the guss asphalt mixture. However, the manual and guideline of AsButon guss asphalt mixture are need. It should be including quality control of production, construction, and maintenance works.

In advanced use, it is need to continue to study on the modified method of Pure-AsButon, and/or usage as modifier additives to straight asphalt.

9. ACKNOWLEDGEMENT

This paper describes cooperation research between IRE and PWRI. The authors acknowledge the support and cooperation of Institute of Road Engineering (IRE, Indonesia) and National Institute for Land and Infrastructure Management (NILIM Japan)

REFERENCES

1. Introduction

Asphalt pavement is quick and easy to construct but vulnerable to heavy traffic loads especially stationary loads. On the other hand, cement concrete pavement has high bearing capacity, but takes curing time that reaches more than two weeks. Short-time pavement technology is eagerly required to construct a new durable road or reinforce existing roads without long-time lane closure to minimize congestion and economic loss. Semi-Rigid pavement is combination of merits between asphalt pavement (flexible) and cement concrete pavement (rigid). Its composition is porous asphalt filled with a special cement grout. This pavement shows not only better bearing capacity with rutting-resistance and wear-resistance, but also resistance against fire, heat, oil and chemical than asphalt pavement. The same types were developed for consideration of scenery and environment. Additional processes such as coloration, shot-blast and cutting can create excellent harmony with surrounding environment and make it look like a natural stone pavement. Furthermore, from the viewpoint of improvement in environment, cement grout including low-carbon cement or water-holding materials can be used, which serve to mitigate heat-island phenomenon and reduce carbon dioxide emission. This paper describes merits and examples of semi-rigid pavement and shows data about the properties.

2. Outline

Long service life, high durability and low-cost pavement that meets economic and environmental requirements are in high demand in many countries. Most of pavement is roughly divided into two types. One is asphalt pavement that is flexible, and quick and easy to construct and repair. Another is cement concrete pavement that is rigid and strong, but takes some time for curing. Semi-rigid Pavement is excellent combination between two pavements, with voids of porous asphalt filled with special cement grout (see Photo 1). It also features multiple-resistance such as rutting-resistance, oil-resistance, fire-resistance, which results in long service life. In addition, it is possible to give good appearance to the pavement by adding a pigment, shot-blasting and cutting by a few millimeters on a surface. Coloration serves to divide lanes on a multi-lanes road and to give good harmony between pavement and the surrounding areas. Shot-blasting and cutting are the processes that make the pavement look like an interlocking or natural-stone pavement. Due to these features described above, semi-rigid pavement has been applied to the following various locations such as,

(1) Locations where a large static load acts
   An intersection, a bus stop, a toll gate, etc.
(2) Locations where a large load is provided
   A taxiway at an airport, a container yard in a harbor, etc.
(3) Locations where excellent appearance is preferred
   A sidewalk, a cycle-road, a park-road, etc.

Photo 1: Surface of Semi-Rigid Pavement

Photo 2: Production of cement grout
Photo 3: Discharge
Photo 4: Injection
2. Ingredient

2-1. Asphalt Mixture
The example of gradation of porous asphalt mixture used for semi-rigid pavement is showed in Table 1. Percentage of voids of the mixture is generally 20 to 28%. Attention should be paid to asphalt content and gradation not to clog the inside voids and make certain that the cement grout easily reaches up to the bottom of the layer.

2-2. Ingredient of Special Cement Grout
The special cement grout is a mixture composed of cement, fly ash, silica sand, and resin-additives, and water (see Table 2). A type of cement is chosen according to a curing time. If a shorter curing time is required for a road maintenance, high-early strength cement (1 day to return to traffic) or ultra-high-early strength cement (3 hours) is applied in place of standard Portland cement (3 days). Weight of each ingredient needs to be measured on the site, however, this process takes labor and time, sometimes causing measurement mistake. A paper-bagged product, which includes all materials, is often used to mitigate the labor during grout-production process. In the process, predetermined number of bags and water corresponding to it is put into a mixer and discharged onto the surface, followed by injection and finishing of the grout (see Photo 2–4).

2-3. Consistency of Special Cement Grout
Consistency of the cement grout is as important as strength to determine a design mix of a grout. It is measured with the funnel-shaped container shown in Figure 1. It is filled up to a reference level with the grout keeping a hole of the bottom covered with a finger. After that, a finger is released from the hole and the grout begins to drop (see Photo 5). The consistency is quantified by the time taken until the grout is discharged from the container by only gravity. Compatibility is needed between workability and strength for the cement grout to easily be injected to the bottom of the layer, and securing enough strength. The design mix is set to take the time between 10 to 14 seconds according to the standard. However, the consistency should be adjusted depending on surface temperature within a range of the standard so that enough workability and permeability can be secured. The time should also be checked not only at a laboratory before the construction but on the site just after production of the cement grout.

2-4. Strength of Cement Grout and Semi-Rigid Pavement
Strength of the grout and the semi-rigid pavement needs to be measured, and a design mix be determined to meet the standard (Table 3). The strength is also be checked using the grout produced on the site for quality control. The compressive strength of more than 5 MPa of the grout is required when a traffic is restored.

3. Property of Semi-Rigid Pavement

3-1. Bending Strength and Compressive Strength
Comparison of bending and compressive strength between a semi-rigid pavement and a dense-graded asphalt, was conducted at temperatures of -10, 0, 10, 20, 40 degrees C. The bending test was carried out with specimens aged 7 days that measure 5cm x 5cm x 16cm, and the cracked remains of the specimens were also used for a compressive test. As shown in the result (see Figure 2, 3), semi-rigid pavement shows larger strength and is less influenced by a temperature than a normal asphalt pavement. It means that semi-rigid pavement has a bearing capacity against a heavy traffic.
3.2 Static-Load-Resistance

Semi-rigid pavement has flexibility as asphalt has, on the other hand it has rigidity similar to cement concrete. Asphalt pavement is vulnerable to a static load that acts at intersections, bus stops, parking lots, etc. Moreover, a steel-crawler leaves behind clear traces damaging the surface. A penetration test was conducted to prove resistance to a static load. Specimens of semi-rigid pavement, dense graded asphalt with modified-asphalt binder and straight-asphalt, which are 7 day aged and measure 7 cm x 7 cm x 7 cm, were given a pressure of 1.0 MPa and 2.0 MPa on a surface by a penetration rod whose diameter is 25.2 mm at temperatures of 10, 20, 30, 40 degrees C. The penetration amount was measured 45 minutes after the start of loading. The result shown in Figure 4 and 5 indicates that the penetration amount of semi-rigid pavement is smaller than that of normal asphalt pavement (dense graded asphalt) proving its static load resistance, and its excellent property stands out at a higher temperature of 60 degrees C and under a larger pressure of 2.0 MPa.

3.3 Dynamic-Load-Resistance

Asphalt pavement is vulnerable not only to static loads but also to dynamic loads. Rutting is a distress that most of road agencies are annoyed with and various measures have been taken from the viewpoint of a property of asphalt binder and a gradation of a mixture. But when a traffic volume of heavy vehicles is large, they don’t become a radical solution. A test called “a wheel tracking test” was conducted to prove a resistance to a dynamic traffic load for semi-rigid pavement, compared with a dense-graded asphalt. The resistance is quantified by dynamic stability of a specimen that measures 300 x 300 x 50mm, which is the number of passing times of the test wheel taken to form the rutting depth of 1 mm at temperature of 60 degrees C (see Photo6). The test wheel is 200 mm diameter, 50mm wide and loaded with weights so that the grounding pressure is 0.63 MPa, 1.37 MPa respectively. The result shown in Figure 6 indicates that a deformation on semi-rigid pavement doesn’t reach 1 mm even after 100,000 passing times with the pressure of 1.37 MPa on the specimens, proving that semi-rigid pavement has excellent dynamic-load resistance.
3-4. Wear-Resistance of Pavement

Wear-Resistance of the pavement was proved by a raveling test. The specimens of semi-rigid pavement and dense-graded asphalt that measures 45cm x 15 cm x 5cm were swatted and damaged by a rotating chain, the conformation of which is specified. The chain is wound around a wheel sized 25cm external diameter and 10 cm in width and set up so that an interval between the wheel and the specimen is 4cm. While the chains are rotating, the specimen goes back and forth at a rate of 66 round trips per a minute. The wear-resistance is quantified by an average abrasion loss at designated measuring cross sections of the specimens. The result in Figure 7 shows better wear-resistance for semi-rigid pavement. The less wear-resistance comes from the strength of the special cement grout injected to voids, leading to prevention of dust generation by traveling vehicles.

![Figure 7: Comparison in Abrasion Loss](image1)

3-5. Wear-Resistance of Cement Grout

The test called “a taper abrasion test” was conducted to evaluate wear-resistance of a cement grout according to JIS A 1453. The specimens of a special cement grout used for semi-rigid pavement, and a normal cement grout sized 10cm in diameter, 1cm thick and 7-day aged, were rotated in touch with a hard rubber disk (see Photo8). The disk is loaded with 10kN and the abrasion loss is measured after 1,000 rotations. The result, as shown in Figure 8, indicates that a special cement grout that includes resin additive has excellent wear-resistance compared to a normal cement grout. It is concluded from the data that wear-resistance of semi-rigid pavement results from not only its structure but also a special cement grout with resin.

![Figure 8: Comparison in Abrasion Loss](image2)

3-6. Heat (Fire)-Resistance

Heat resistance is required in a place where a high temperature acts on such as a surface or floor on which machinery is installed. Although asphalt pavement is vulnerable to heat and fire due to its property of asphalt binder, the resistance can be increased by introduction of semi-rigid pavement. The resistance was proved by a heating test with the specimens of dense graded asphalt and semi-rigid pavement (7 day aged) sized 10 x 15 x 4cm. The specimens were heated in an oven up to 100, 200, 300 degrees C for 4 hours, and the heat-resistance was quantified by bending strength measured after cooling to ambient temperature. The result, as showed in Figure 9, indicates that the resistance of semi-rigid pavement to heat (fire) is much larger than dense graded asphalt.

3-7. Oil-Resistance

Oil is sometimes leaked from a vehicle on a road especially at an intersection or a gas stand. Although asphalt pavement is easily damaged due to solubility to oil, the resistance can be increased by introduction of semi-rigid pavement. The resistance was proved by an oil-impregnation test. The specimens of dense graded asphalt and semi-rigid pavement (7 day aged) sized 10cm x 15cm x 4cm was impregnated in a light oil for 60, 120, and 180 minutes respectively. The oil-resistance was quantified by a bending strength after oil-impregnation. As showed in Figure 10, semi-rigid pavement indicates excellent resistance compared to dense graded asphalt almost not affected by oil.
3-8. Skid-Resistance
There was a concern that semi-rigid pavement has less skid-resistance than normal asphalt pavement. Skid–resistance was measured to compare two pavements by a “Portable Skid Resistance Tester”. As shown in Figure 11, there is little difference in the value of BPN (British Pendulum Number) on both dry and wet conditions. However, the value on wet condition depends on finishing on a surface. When much cement grout is left above a top of aggregate, it causes reduction in skid-resistance. Therefore, it is required to leave as little cement grout on a surface as possible with such tools as a rubber lake and a broom.

4. Consideration of Environment

4-1. Improvement in Appearance
Semi-rigid pavement can be applied as colored pavement to a sidewalk or a park road by adding a pigment to a cement grout. In addition, it can be made look like a natural stone by the process of shot-blast, grinding, cutting a few millimeters on a surface. These processes can extend the application of semi-rigid pavement to various roads.

4-2. Water-Retention Asphalt Pavement
(1) Overview
Normal asphalt pavement easily absorbs heat from infra-red ray of the sunlight due to its dark color, and the surface-temperature can reach up to 60 degrees C in the mid-summer. It is widely known that the rise in surface-temperature can contribute to heat-island phenomenon in an urban area. Water-retention asphalt pavement is similar to a semi-rigid pavement, with is a porous asphalt whose voids are filled with a cement grout, except that the grout includes inorganic mineral water-retentive materials. The structure of the water-retention pavement constructed is shown in Figure 12.

(2) Property of Cement Grout
When a rain falls, a cement grout injected in voids of porous asphalt absorbs water which corresponds to 47% of the volume of the cement grout. Water gradually vaporizes and draws heat from a surface. Strength of the cement grout is less than the one used for semi-rigid pavement due to capacity to absorb much water. However, the resistance evaluated
above for semi-rigid pavement is much more than normal asphalt pavement. Property of the cement grout is shown in Table 8. Largest water absorption rate is calculated according to the equation 1.

![Schematic of the pavement](image)

**Figure12: Schematic of the pavement**

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table4: Example of Cement Grout**

<table>
<thead>
<tr>
<th>Element</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Largest Water Consumption Rate</td>
<td>47</td>
<td>%</td>
</tr>
<tr>
<td>Dynamic Stability</td>
<td>8,400</td>
<td>Times/ton</td>
</tr>
<tr>
<td>Abrasion Loss of the pavement</td>
<td>1.1</td>
<td>cm³</td>
</tr>
<tr>
<td>Compressive Strength of Cement Grout (7 days aged)</td>
<td>5.2</td>
<td>MPa</td>
</tr>
</tbody>
</table>

\[
\text{Largest Water Consumption Rate} = \frac{\text{Surface Dry Weight} - \text{Dry Weight}}{\text{Volume of Sample}}
\]  
(Equation 1)

(3) Inhibitory Effect against rise in Surface-Temperature

Data of surface-temperature measured on both water-retention pavement and normal asphalt pavement is shown in Figure13. The result indicates that the water-retention pavement is about 15 degrees C lower than normal asphalt pavement at maximum. The temperature difference gradually between two pavements gets small after a few days without raining. However, as long as rain water is supplied periodically, rise in the surface-temperature is well suppressed.

4-3. Low-carbon semi-rigid pavement

Reduction in CO₂ emission is required to ease global warming in all industries, and a construction industry is no exception. A cement grout for low-carbon semi-rigid pavement includes low-carbon cement that is manufactured from a recycled cement or by changing a production method of cement so as to emit less CO₂. The pavement can reduce CO₂ emissions by about 21% for standard Portland cement type, and about 53% for high-early strength type, as shown in Figure 14. This reduction is equivalent to CO₂ emission from combustion of gasoline of about 1,000 litters and 3,000 litters respectively.

![Comparison in CO₂ Emission](image)

**Figure14: Comparison in CO₂ Emission**

5. Summary

Semi-rigid pavement has excellent combination between asphalt pavement and cement concrete pavement. Among the main features are high strength and durability. The property of oil- and heat-resistance extends its application to not only roads but also floors in a factory and a gas stand and so on. Moreover, it is quick to construct and return to the traffic unlike cement concrete, therefore this pavement is suitable for repair and maintenance work. It cost the cement grout, but can extend a service-life and reduce life-cycle-cost. Three types of cements are available depending on curing time, and the time ranges from 3 days to 3 hours. It also can improve appearance keeping good harmony with scenery of a surrounding area by such processes as shot-blasting or cutting on a surface to look like a natural stone. However, the achievement area has not been on the rise in recent years in Japan. It looks like that the merit of the pavement may not be understood enough. We hope that this pavement is

Reference

Examining Factors Affecting the Severity of Run-off-Road Crashes:

Abu Dhabi Case Study

Mohamed Shawky, Ph.D.,*,1,2
Hany M. Hassan, Ph.D.,1,2
Atef M. Garib, Ph.D.,
Hussain A. Al-Harthei,

1 Traffic and Patrols Directorate
Abu Dhabi Police, Abu Dhabi,
United Arab Emirates

2 Department of Civil Engineering,
Faculty of Engineering,
Ain Shams University,
Cairo, Egypt

*Corresponding Author:
Mohamed Shawky, Ph.D.,
Tel: +971 561236607
Email address: m_shawky132@hotmail.com

Manuscript prepared for presentation at the 1st IRF Asia Regional Congress
(November 17-19, 2014, Bali, Indonesia)

September 2014
Abstract

The crashes severity has been extensively investigated in numerous prior studies. However the number of studies that addressed the severity of the run-of-road (ROR) crashes is relatively low compared to other crash types. In the Emirate of Abu Dhabi (AD), the ROR crashes represent about 22% of the total severe crashes and led to the same percentage of total crash fatalities. Despite these facts, the factors contributing to the occurrence and severity of ROR crashes in AD have not been explicitly addressed in prior studies. This study aims to explore the characteristics of at-fault drivers involving in ROR crashes in AD. In addition, it aims to identify and quantify the factors affecting the severity of ROR crashes occurring such as drivers’ factors, road site characteristics, vehicles and environmental features.

Logistic regression model was developed. The results indicated that driver’s factors (nationality, education, carelessness and speeding), vehicle’s factors (vehicle’s defects and vehicle type), and road’s factors (crash location: at non-intersection locations vs at intersections) were the significant factors that affect the severity of ROR crashes occurring in AD. Countermeasures to improve traffic safety and reduce numbers and severity of ROR crashes are also discussed.

Key Words: Crash severity, Run-off-Road crashes, traffic Safety, Abu Dhabi, Logistic regression
1. INTRODUCTION

Single-vehicle crash is defined as a crash that occurs when a vehicle hits an object in or out of the roadway. The majority of the single-vehicle crashes are considered as run-off-road (ROR) crashes or roadway departure crashes which occur after a vehicle leaves the designated roadway and hit with a fixed object or pedestrian on the roadside/median or hit another vehicle travelling in the opposite direction or overturned. Previous studies indicated that 30% to 55% of rural fatalities are due to ROR crashes (usually due to hitting fixed object). For example, there were 15,307 ROR fatal vehicle crashes in 2011, which accounted for 51% of all fatal crashes in the United State of America (USA) and resulting into 16,948 fatalities (FHWA, 2013). In addition, David et al. (2008) reported that about 32% of total traffic fatalities resulted from the ROR crashes in USA. However, this ratio reached 52.5% in the state of Ohio based on the crash data period 2007-2011 (NHTSA, 2013).

In the Emirate of Abu Dhabi (AD), ROR crashes constitute a significant contribution of serious crashes (i.e., crashes that resulted in at least one injury or fatality). Table 1 shows the total number of serious crashes that is occurred in AD from 2007 to 2013. Also, it shows the number and percentage of the single-vehicle and ROR crashes. On the other hand, Table 2 shows the number of total crash fatalities and the fatalities resulted from the ROR crashes at the same period. These tables indicate that the road safety in AD has been improved in general during the last few years. However, the percentage of ROR crashes and its fatalities remained around its value without improvement. Furthermore, the crash statistics in AD revealed that the majority (about 62%) of ROR crashes occurring in AD were overturned-vehicle crashes. While about 38% of ROR crashes in AD occurred due to hitting objects. Additionally, it was found that the severity of the ROR crashes in AD is significantly higher (156 fatality/1000 crashes) than the severity of multi-vehicle crashes (112 fatality/1000 crashes).

Based on the facts mentioned above, the main objective of this study is to provide in-depth insights regarding the nature, main causes and the factors associated with the severity of ROR crashes in AD. It also aims to provide better understanding of the characteristics of at-fault drivers involving in ROR crashes occurring in AD.

| Table 1: Distribution of serious, single-vehicle and ROR Crashes in AD from 2007 to 2013 |
|--------------------------------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| Year                                | 2007  | 2008  | 2009  | 2010  | 2011  | 2012  | 2013  | Total |
| Total number of serious crashes     | 2646  | 2930  | 3086  | 2537  | 2283  | 2056  | 2071  | 17609 |
| Single-vehicle crashes              |       |       |       |       |       |       |       |       |
| number                              | 1485  | 1586  | 1665  | 1373  | 1239  | 1037  | 969   | 9354  |
| %                                   | 56%   | 54%   | 54%   | 54%   | 54%   | 50%   | 47%   | 53%   |
| ROR crashes                         |       |       |       |       |       |       |       |       |
| number                              | 549   | 543   | 676   | 577   | 575   | 471   | 428   | 3819  |
| %                                   | 21%   | 19%   | 22%   | 23%   | 25%   | 23%   | 21%   | 22%   |

| Table 2: Distribution of crash fatalities in AD from 2007 to 2013 |
|--------------------------------------|-------|-------|-------|-------|-------|-------|-------|-------|
| Year                                | 2007  | 2008  | 2009  | 2010  | 2011  | 2012  | 2013  | Total |
| Total number of crash fatalities    | 365   | 376   | 409   | 376   | 334   | 271   | 289   | 2420  |
| Fatalities from ROR crashes         |       |       |       |       |       |       |       |       |
| number                              | 82    | 75    | 93    | 77    | 74    | 68    | 60    | 529   |
| %                                   | 23%   | 20%   | 23%   | 21%   | 22%   | 25%   | 21%   | 22%   |
2. BACKGROUND

Considering previous studies (for example, Omar 2013; Peter et al. 2011; Jung et al. 2010; NHTSA 2009; Gray et al. 2008; Abdel-Aty 2003; Khattak and Targa, 2004; Chen and Jovanis 2000), the factors that affect the severity of the traffic crashes can be classified into five classes: 1) driver characteristics (age and gender); 2) driver behavior factors (driving under the influence of drugs or alcohol use, speeding, fatigue and drowsiness, using cell phones and seat belt while driving, etc.); 3) highway factors (speed limit, number of lane, lane and shoulder width, roadside objects, number of lanes, presents of the vertical and horizontal curves, pavement condition, etc.); 4) environmental factors (weather condition, road surface condition, time of day and day of week); 5) traffic characteristics (traffic volume and truck percentage).

Numerous types of statistical models have been employed in the previous studies in order to analyze the severity of traffic crashes. Mannerling and Bahat (2014) conducted a comprehensive review of the methodological approaches that were previously applied for modeling the severity of traffic crashes. About 26 statistical models were listed and discussed in that study. The most common modeling approaches that were used in theses previous researches were the logistic regressions, binary logit, ordered probit, multinomial logit, nested logit model and mixed logit models (i.e., Fan and Dominique, 2011; Abdel-Aty and Keller, 2005; Haleem and Abdel-Aty, 2010; Kim 2008; Yasmim and Eluru, 2013).

For instance, Richardson et al. (1996) indicated that young drivers were more likely to experience ROR crashes while older drivers were more likely to experience on road crashes. Many other studies showed that male drivers have a higher probability of being involved in ROR crashes than female (i.e., Chen and Chen, 2011; Joon et al., 2010). In addition, Liu and Subramanian (2009) applied the regression analysis to identify the contributing factors affecting the occurrence of ROR crashes. The results indicated that drivers’ behavior factors (fatigue, drive under the influence of alcohol, speeding) followed by road factors (curve design parameters, rural roads, and high posted speed) increase the crash severity.

Lee and Mannerling (2002) investigated the impact of road side feature on the severity of the ROR crashed using nested logit model. In this study, detailed roadside feature variables (i.e., guardrails, slopes, trees, fences, poles, etc.) have been investigated along with the ordinary features of the roads, drivers, and vehicles and environmental. The results showed that high posted speed, narrow shoulder, night time, alcohol, older driver, horizontal curves and bridges presence, median, catch basin, cut side slope, guardrail increase the crash severity.

Spainhour et al. (2008) studied the fatal ROR crashes using binary regression analysis by testing 23 explanatory variable including driver, road, vehicle and environmental factors. It was found that the presence of rumble strips, bad weather, rural locations, incapacitated drivers, running off the road to the left or straight are positively associated with over-correction. In addition, the results showed that paved or curbed shoulders, wet or slippery roads and larger vehicles are not significantly affecting the ROR crashes.

Sunanda and Uttara (2014) developed three different binary logit models for three levels of ROR crash severity. These models were established by using 72,181 crash records and 43 explanatory variables. The results showed that 20 variables appeared to be positively associated with ROR crashes. These variables are driver related factors such as driver ejection, older driver, alcohol involvement, license state, drivers at-fault, medical condition of the drivers; road related variables such as speed, asphalt road surface, dry
road condition; crashes occurring between 6 pm and midnight, daylight as environment related factors; vehicle related factors such as SUVs, motorcycles, vehicle destroyed, vehicle disabled, vehicle straight, and vehicle passing; and tree and ditch as fixed objects types. They also found that the usage of safety equipment, straight and level road, and vehicle registration have a decreasing tendency towards the crash severity for all three models. The positive effect of using the seat belt in reducing crash severity has been also proved by using probit logit model (Ratnayake, 2006; Abay et al., 2013).

In the current study, logistic regression approach was implemented to indentify the impacts of all significant factors affecting the severity of ROR crashes in AD (binary target variable: fatal vs injury crash).

3. DATA DESCRIPTION

Data from 3,819 ROR severe crashes that occurred in AD from 2007 to 2013 was employed in this study. Severe crashes are defined as the crashes that resulted in at least one injury or fatality. It was not possible for the present study to obtain data for the property damage only (PDO) crashes and hence there were not included in the analysis presented in the following sections.

The database of AD traffic police includes full information about the at-fault-drivers, vehicles characteristics, causalities, road and environmental factors. In AD crash report, the severities of traffic crashes are classified into five levels; fatal, severe injury, medium injury, slight injury and PDO.

It is worth mentioning that the licensed drivers in AD have a unique compassion where more than one hundred different nationalities live and work there. Emirates drivers represent 13%, Asian drivers represent 47%, and Arab represents 27%, others represents13% of total driving licenses in AD. Male drivers represent 85% of total number of total driving licenses and 92% has age less than 45 years old.

Table 3 shows the characteristics of at-fault drivers being involved in ROR crashes that occurred in AD during the period from 2007 to 2013. It was found that males, young, low educated, Emirati and Asian drivers contribute to ROR crashes more than females, old, high educated and drivers from other nationalities. It also shows that the majority (92%) of at-fault drivers being involved in severe ROR crashes in AD were male drivers. In addition, the results indicate that about half (50%) of at-fault drivers being involved in ROR crashes aged 30 years old or less.

Furthermore, more than two thirds (72%) of those at-fault drivers were Emirati and Asian. The findings point out that Emirati and Asian drivers have the highest and same contribution percentage (36% for each) despite their different representation in total driving license in AD (i.e., Emirati and Asian drivers represent about 13% and 47% of the total licensed drivers in AD, respectively). This result indicates that Emirati drivers are overrepresented in ROR crashes occurring in AD. Regarding education, it was found that more than half of the at-fault drivers being involved in those crashes (55%) have no education degrees (only read and write).

Table 4 illustrates the characteristics of ROR crashes occurring in AD between 2007 and 2013. The results indicated that the majority of ROR crashes occurred on rural roads (79%), at non intersection locations (77%), at clear weather conditions (95%), and at high posted speed roads (61%). Regarding crash causes, it was found that sudden lane change (47%), speeding (16%), vehicle/road defects (10%) were the main causes of severe ROR crashes occurring in AD. Other factors that may affect the ROR
crash severity are not included due to the limitation of the available data such as the existence of road side fixed objectives (barriers, polls, trees, etc.) and the presence of horizontal/vertical curves

Table 3: At-fault-drivers Characteristics of being involved in ROR crashes

<table>
<thead>
<tr>
<th>Variable classifications</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gender</td>
<td></td>
</tr>
<tr>
<td>Male</td>
<td>92%</td>
</tr>
<tr>
<td>Female</td>
<td>8%</td>
</tr>
<tr>
<td>Age Group</td>
<td></td>
</tr>
<tr>
<td>18-24</td>
<td>27%</td>
</tr>
<tr>
<td>25-30</td>
<td>23%</td>
</tr>
<tr>
<td>31-40</td>
<td>27%</td>
</tr>
<tr>
<td>41-50</td>
<td>14%</td>
</tr>
<tr>
<td>51-60</td>
<td>7%</td>
</tr>
<tr>
<td>&gt;60</td>
<td>2%</td>
</tr>
<tr>
<td>Nationality</td>
<td></td>
</tr>
<tr>
<td>UAE</td>
<td>36%</td>
</tr>
<tr>
<td>GCC*</td>
<td>2%</td>
</tr>
<tr>
<td>Other Arabian</td>
<td>23%</td>
</tr>
<tr>
<td>Asian</td>
<td>36%</td>
</tr>
<tr>
<td>Other nationalities</td>
<td>3%</td>
</tr>
<tr>
<td>Education level</td>
<td></td>
</tr>
<tr>
<td>Read / Write</td>
<td>55%</td>
</tr>
<tr>
<td>School</td>
<td>36%</td>
</tr>
<tr>
<td>High educated</td>
<td>9%</td>
</tr>
<tr>
<td>Marital Status</td>
<td></td>
</tr>
<tr>
<td>Married</td>
<td>44%</td>
</tr>
<tr>
<td>Single</td>
<td>47%</td>
</tr>
<tr>
<td>Other</td>
<td>9%</td>
</tr>
</tbody>
</table>

* GCC: Gulf Country Council nationalities
Table 4: Characteristics of ROR crashes occurring in AD 2007-2013

<table>
<thead>
<tr>
<th>Variable classifications</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Road features</strong></td>
<td></td>
</tr>
<tr>
<td>Road Type</td>
<td>Rural 79%</td>
</tr>
<tr>
<td></td>
<td>Urban 21%</td>
</tr>
<tr>
<td>Speed Limit</td>
<td>≤ 60 19%</td>
</tr>
<tr>
<td></td>
<td>80 19%</td>
</tr>
<tr>
<td></td>
<td>100 27%</td>
</tr>
<tr>
<td></td>
<td>≥ 120 34%</td>
</tr>
<tr>
<td>Intersection-related</td>
<td>Close or at intersection/junction 23%</td>
</tr>
<tr>
<td></td>
<td>Road segment 77%</td>
</tr>
<tr>
<td>Number of lanes</td>
<td>≤ 2 lanes 49%</td>
</tr>
<tr>
<td></td>
<td>3 lanes 37%</td>
</tr>
<tr>
<td></td>
<td>≥ 4 lanes 14%</td>
</tr>
<tr>
<td><strong>Vehicle feature</strong></td>
<td></td>
</tr>
<tr>
<td>Vehicle type</td>
<td>passenger car 83%</td>
</tr>
<tr>
<td></td>
<td>Heavy vehicle 17%</td>
</tr>
<tr>
<td><strong>Environmental</strong></td>
<td></td>
</tr>
<tr>
<td>Light condition</td>
<td>Enough light (day time/night with high illumination) 89%</td>
</tr>
<tr>
<td></td>
<td>Other (night with low or without illumination) 11%</td>
</tr>
<tr>
<td>Weather</td>
<td>Clear 95%</td>
</tr>
<tr>
<td></td>
<td>Unclear 5%</td>
</tr>
<tr>
<td>Surface road condition</td>
<td>Dry 88%</td>
</tr>
<tr>
<td></td>
<td>Other (wet/sand) 12%</td>
</tr>
<tr>
<td>Day Time</td>
<td>Morning 29%</td>
</tr>
<tr>
<td></td>
<td>at Noon 22%</td>
</tr>
<tr>
<td></td>
<td>After noon 11%</td>
</tr>
<tr>
<td></td>
<td>Evening 38%</td>
</tr>
<tr>
<td>Day of week</td>
<td>Weekend days 31%</td>
</tr>
<tr>
<td></td>
<td>Working days 69%</td>
</tr>
<tr>
<td>Land use</td>
<td>Residential/commercial 28%</td>
</tr>
<tr>
<td></td>
<td>Public service 9%</td>
</tr>
<tr>
<td></td>
<td>Other 63%</td>
</tr>
<tr>
<td><strong>Crash Feature</strong></td>
<td></td>
</tr>
<tr>
<td>Crash Causes</td>
<td>Sudden lane change 47%</td>
</tr>
<tr>
<td></td>
<td>Speeding 16%</td>
</tr>
<tr>
<td></td>
<td>Alcohol 4%</td>
</tr>
<tr>
<td></td>
<td>Sleepy 2%</td>
</tr>
<tr>
<td></td>
<td>Other drivers' faults 21%</td>
</tr>
<tr>
<td></td>
<td>Vehicle/road defect 10%</td>
</tr>
<tr>
<td></td>
<td>Bad weather 0.2%</td>
</tr>
<tr>
<td>Crash type</td>
<td>Hit object out of the carriageway 31%</td>
</tr>
<tr>
<td></td>
<td>Turnover 69%</td>
</tr>
</tbody>
</table>
4. MODELING CONTRIBUTING FACTORS AFFECTING SEVERITY OF ROR CRASHES

This section explains the modeling process that was performed in this study to identify the relationships between the injury severity of ROR crashes and the explanatory variables (shown in Tables 3 and 4).

4.1. Identifying Significant Variables

Prior to modeling process, Cochran–Mantel–Haenszel (CMH) test was used to analyze the association between the binary target variable (injury severity) and every explanatory variable in data set. Table 5 illustrates a summary of all variables that were found to have significant associations with the severity of ROR crashes in AD.

As shown in the table, the factors that were found to have significant association with the severity of traffic crashes in AD are: (1) driver’s factors (carelessness, speeding, age, nationality and education), (2) vehicle’s factors (defects in vehicles and type of vehicle), (3) road’s features (whether the crash occurs at non intersection locations and road speed).

Further discussion regarding the interpretation of these significant factors is presented in the results and discussion section.

Table 5: Summary of the factors associated with the severity of traffic crashes

<table>
<thead>
<tr>
<th>Factors</th>
<th>DF</th>
<th>Value</th>
<th>P Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carelessness</td>
<td>1</td>
<td>15.0523</td>
<td>0.0001*</td>
</tr>
<tr>
<td>Speeding</td>
<td>1</td>
<td>30.7307</td>
<td>&lt;.0001*</td>
</tr>
<tr>
<td>Age (young drivers vs others)</td>
<td>1</td>
<td>4.2170</td>
<td>0.0400*</td>
</tr>
<tr>
<td>Nationality (Emirati vs others)</td>
<td>1</td>
<td>10.2824</td>
<td>0.0013*</td>
</tr>
<tr>
<td>Education (High schools or less vs others)</td>
<td>1</td>
<td>5.3070</td>
<td>0.0424*</td>
</tr>
<tr>
<td>Vehicle defects</td>
<td>1</td>
<td>4.6529</td>
<td>0.0310*</td>
</tr>
<tr>
<td>Vehicle type</td>
<td>1</td>
<td>40.4324</td>
<td>&lt;.0001*</td>
</tr>
<tr>
<td>Crash location (at non-intersections vs. others)</td>
<td>1</td>
<td>3.1164</td>
<td>0.0775**</td>
</tr>
<tr>
<td>Road speed</td>
<td>5</td>
<td>11.9380</td>
<td>0.0356*</td>
</tr>
</tbody>
</table>

* Significant at $\alpha = 0.05$, ** Significant at $\alpha = 0.10$

4.2. Model Development

The binomial logistic regression (often referred to simply as logistic regression) predicts the probability that an observation falls into one of two categories of a dichotomous dependent variable based on one or more independent variables that can be either continuous or categorical.

It has been extensively used in many disciplines including medical, engineering, social science, traffic safety and human factors.

In this study, binary logistic regression approach was developed to identify and quantify the factors affecting the severity of ROR crashes in AD (binary variable: fatal crash vs injury crash). The logistic regression model can be written in the following form (Agresti, 2002):
\[ E(Y/X) = \pi(x) = \frac{e^{\beta_0 + \beta_1 x}}{1 + e^{\beta_0 + \beta_1 x}} \] (1)

Where the transformation of the \( \pi(x) \) logistic function is known as the logit transformation:

\[ g(x) = \ln \left[ \frac{\pi(x)}{1 - \pi(x)} \right] \] (2)

The logistic regression model estimated in this study can be expressed as follows:

\[ P(\text{Fatal crash}) = \pi(x) = \frac{e^{\theta(x)}}{1 + e^{\theta(x)}} \] (3)

Where, \( g(x) \) is the function of independent variables:

\[ g(x) = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \cdots + \beta_n x_n \] (4)

5. RESULTS AND DISCUSSION

In this study, logistic regression approach was developed using the Statistical Analysis System (SAS) statistical software, procedure: logistic. The significant factors that found to have significant association with severity of traffic crashes in AD (shown in Table 5) were used as inputs when developing the model. Table 6 illustrates the estimation results and model fit statistics of the best obtained model after using different automatic search technique (i.e., backward, forward, stepwise).

The findings shown in Table 6 indicate that driver’s factors (nationality, education, speeding and carelessness), vehicle’s factors (defects in vehicles and vehicle type) and road’s features (crash location: whether the crash occurs at non-intersection locations or at intersections) were the significant variables affecting the severity of ROR crashes. It is worth mentioning that a positive coefficient of a variable implies that the increase of the variable would increase the severity of crashes (i.e., lead to being involved in fatal crash).

In addition, the odds (hazard) ratio of these significant variables is provided in Table 6. Odds ratio is an estimate of the expected change in the risk ratio of having a fatal vs. non-fatal (injury) crash per unit change in the corresponding explanatory factor.

These results revealed that committing certain aberrant driving behaviors while driving (carelessness and speeding) significantly affect the severity of ROR crashes. A hazard ratio of 0.664 corresponding to the variable carelessness means that the odds of being involved in a fatal ROR crash due to carelessness is about 34% lower than those occurring due to other crash reasons (i.e., speeding, sudden lane change, tailgating, etc.). In other words, carelessness increases the probability of non-fatal (injury) crashes. In addition, the results indicate that the likelihood of being involved in a fatal crash due to speeding is about 2.0 times higher than those occurring due to other crash reasons (i.e., distraction, running red lights). In addition, the findings pointed out that the likelihood of Emirate drivers for being involved in fatal ROR crashes is about 26% higher than other nationalities. Moreover, the results revealed that the probability of low educated drivers (high school or lower) for being involved in fatal ROR crashes in AD is about 22% higher than other drivers (having university degrees or higher).

Regarding vehicle type, as expected it was found that the involvement of passenger cars in traffic collisions increase the probability of having fatal ROR crash by 2.8 times higher than other larger cars.
such as SUVs/trucks. Also, the findings showed that the likelihood of being involved in fatal crashes due to defects in vehicles is about 49% higher than other crash causes.

Moreover, road factors affected the severity of ROR crashes in AD. The results showed that the odds of having fatal ROR crashes at non-intersection locations is about 25% higher than those occurring at intersections (this result is consistent with Al-Ghamdi, 2002).

Considering the results of prior studies, it can be realized that the impact of the driver’s factors (i.e., carelessness and speeding), vehicle factors and road factors on crash severity are consistent with the findings of previous researches (i.e., Sundra and Uttara, 2014; Liu and Subramanian, 2009; Spainhour et al., 2008). However, drivers’ age and gender showed insignificant impact on the ROR crash severity in AD. This is in contrast with the findings of prior studies. This can be explained due to the unique composition of the drivers’ population in AD as the majority of licensed drivers are young (i.e., over 92% under 45 years old and 0.5% over 60 years old). In addition, male drivers represent 85% of total licensed drivers in AD. It is worth mentioning that two new variables (i.e., education and nationality) showed significant impact on ROR crash severity.

The model fit statistics as well as the association of predicted probabilities and observed responses are provided in Table 6. Additionally, Figure 1 illustrates the ROR curve of the fitted logistic regression model. As shown in the figure, the area under the curve was 0.6335.

| Table 6: Estimation results and model fit statistics of logistic regression model |
| Analysis of Maximum Likelihood Estimates | Odds Ratio |
| Parameter | DF | Estimate | Standard Error | Wald Chi-Square | Pr > ChiSq | Estimate | 95% Wald Confidence Limits |
| Intercept | 1 | -3.3715 | 0.2154 | 244.9510 | <.0001 | - | - | - |
| Driver factors | | | | | | | | |
| Nationality | 1 | 0.2297 | 0.1066 | 4.6437 | 0.0312 | 1.258 | 1.021 | 1.551 |
| Education | 1 | 0.1985 | 0.1074 | 3.4135 | 0.0647 | 1.220 | 0.988 | 1.505 |
| Speeding | 1 | 0.6815 | 0.1281 | 28.2907 | <.0001 | 1.977 | 1.538 | 2.541 |
| Carelessness | 1 | -0.4102 | 0.2027 | 4.0957 | 0.0430 | 0.664 | 0.446 | 0.987 |
| Vehicle factors | | | | | | | | |
| Vehicle Defects | 1 | 0.4002 | 0.1734 | 5.3303 | 0.0210 | 1.492 | 1.062 | 2.096 |
| Vehicle Type | 1 | 1.0144 | 0.1867 | 29.5272 | <.0001 | 2.758 | 1.913 | 3.976 |
| Road factors | | | | | | | | |
| Crash location | 1 | 0.2256 | 0.1109 | 4.1425 | 0.0418 | 1.253 | 1.008 | 1.557 |
| Model Fit Statistics | | | | | | | | |
| Criterion | Intercept Only | Intercept and Covariates |
| AIC | 2795.475 | 2711.231 |
| SC | 2801.722 | 2761.213 |
| -2 Log L | 2793.475 | 2695.231 |
| Association of Predicted Probabilities and Observed Responses | | | |
| Percent Concordant | 60.8 | Somers’ D | 0.267 |
| Percent Discordant | 34.1 | Gamma | 0.282 |
| Percent Tied | 5.2 | Tau-a | 0.056 |
| Pairs | 1533528 | c | 0.634 |
6. CONCLUSION AND RECOMMENDATIONS

The Emirate of Abu Dhabi (AD) has a unique population composition where more than 100 different nationalities live and work there. In this regards, Emirati (local) drivers represent only about 13% of total licensed drivers in AD. Additionally, males and drivers aged 18 to 25 years represent approximately 85% and 92% of total licensed drivers in AD, respectively. Moreover, the statistics indicated that about 22% of the total serious crashes that occurred in AD from 2007 to 2013 were ROR crashes. Those ROR crashes resulted in about 22% of total fatalities and 25% of the total serious injuries in AD. Despite these figures and the uniqueness of licensed drivers’ composition in AD, the factors contributing to the occurrence and severity of ROR crashes in AD have not been explicitly addressed in any prior studies.

Thus, the main objectives of this study were to: (1) examine the characteristics of at-fault drivers being involved in ROR crashes in AD, (2) explore the nature and main causes of those crashes, and (3) identify the impacts of significant variables affecting the severity of ROR crashes occurring in AD.

To achieve these objectives, a total of 3,819 severe ROR crashes that occurred in AD from 2007 to 2013 were employed and used in the analysis. Logistic regression model approach was developed.

The findings revealed that males, young and low educated drivers contribute more in ROR crashes than females, old and high educated drivers. Moreover, the findings indicated that Emirati drivers are overrepresented in ROR crashes in AD. It was found that 36% of at-fault drivers being involved in ROR crashes in AD were Emirati drivers while they represent only about 13% of the total licensed drivers. The results also showed that the majority of ROR crashes occurred on rural roads, straight road segments and high posted speed roads. Moreover, the findings revealed that sudden lane change, speeding, road and vehicle defects were the main causes for ROR severe crashes occurring in AD.
In addition, the results of logistic regression model indicated that driver’s factors (nationality, education, speeding and carelessness), vehicle’s factors (defects in vehicles and vehicle type) and road’s features (crash location: whether the crash occurs at non-intersection locations or at intersections) were the significant variables affecting the severity of ROR crashes in AD. This study provides traffic safety authorities in AD with valuable insights on how to improve traffic safety on AD roads and consequently reduce the numbers and severity of ROR crashes which may support and improve the overall traffic safety performance in AD. In this regards, traffic safety authorities in AD are recommended to focus their future efforts on improving two main aspects: education and enforcement.

Concerning education, future education courses and/or campaigns should focus their attention on targeting males, young, low educated, Emirati and Asian drivers as they are more likely to be involved in ROR crashes in AD. These education courses/campaigns should emphasize the negative effects of committing certain aberrant driving behaviour including speeding, carelessness and sudden lane change and the strong relationship between them and increasing the risk of being involved in severe traffic crashes. More awareness/campaigns regarding the periodical check and maintenance of vehicle especially tires are required. In addition, a change in driver’s attitudes might be achieved through compulsory refresher courses for offenders, improving existing driving lessons for beginners and continuous awareness campaigns on television and advanced social media (i.e., Facebook, Twitter and Instagram).

In addition, traffic safety agencies in AD are advised to offer more speed enforcement on straight segments of rural roads to improve drivers’ compliance with posted speed. In this regards, they are advised to expand the use of automated speed enforcement techniques on rural roads through using fixed, mobile and point-to-point speed radars. Furthermore, deterrence through strict penalties for repeat offenders might help in improving the aberrant behaviors of drivers in AD.

7. ACKNOWLEDGEMENT

The authors would like to thank the traffic and patrols directorate, Abu Dhabi police for providing the required data in this study.
8. References


**PAPER TITLE**  
(CO$_2$ IMPACT OF 2 WHEELERS IN ASIAN COUNTRIES)

<table>
<thead>
<tr>
<th>TRACK</th>
</tr>
</thead>
</table>

| AUTHOR  
(Capitalize Family Name) | POSITION | ORGANIZATION | COUNTRY |
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Yosuke NAGAHAMA</td>
<td>Researcher</td>
<td>Road Environment Division, National Institute for Land and Infrastructure Management</td>
<td>Japan</td>
</tr>
</tbody>
</table>

| CO-AUTHOR(S)  
(Capitalize Family Name) | POSITION | ORGANIZATION | COUNTRY |
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Shinri SONE</td>
<td>Head</td>
<td>International Research Division, National Institute for Land and Infrastructure Management</td>
<td>Japan</td>
</tr>
<tr>
<td>Agah M. MULYADI</td>
<td>Researcher</td>
<td>Traffic Engineering and Road Environment Division, Institute of Road Engineering</td>
<td>Indonesia</td>
</tr>
</tbody>
</table>

**E-MAIL**  
(for correspondence) do-kan@nilim.go.jp

**KEYWORDS:**  
carbon dioxide, 2 wheeler

**ABSTRACT:**

National Institute of Land and Infrastructure Management in Japan (NILIM) and Institute of Road Engineering in Indonesia (IRE) have jointly studied on "CO$_2$ impact of 2 wheelers in Asian Countries". This paper shows estimation of CO$_2$ emission from road transport in Asian countries, Taking into account of Modal shift from 2 wheelers to 4wheelers.

In the most of Asian Countries, 2 wheelers are the most popular vehicles. In Japan, Europe, US, 2 wheelers are rare. It is easily estimated that 2 wheelers transport will be changed to 4 wheelers transport in Asian countries. 4 wheelers are safer than 2 wheelers, however emits CO$_2$ more.

This paper shows the result of NILIM-IRE Joint Research, which is estimation of Total CO$_2$ emission of road transport according to several scenarios of 2 wheelers ratio.
CO₂ IMPACT OF 2 WHEELERS IN ASIAN COUNTRIES

Yosuke NAGAHAMA¹, Shinri SONE², Agah M. MULYADI³

¹ National Institute for Land and Infrastructure Management (NILIM), Tsukuba, JAPAN
² National Institute for Land and Infrastructure Management (NILIM), Tsukuba, JAPAN
³ Institute of Road Engineering (IRE), Bandung, INDONESIA

Email for correspondence: do-kan@nilim.go.jp

1 INTRODUCTION

Most of countries, CO₂ emissions from the transport sector account for approximately 10–30% of total emissions (International Road Federation 2010). Furthermore, road traffic is responsible for most CO₂ emissions from the transport sector. Whereas 4 wheelers account for the majority of road traffic modes in European countries, the United States, and Japan, 2 wheelers account for the overwhelming share of road traffic in Asian countries other than Japan and South Korea (see Figure 1). Although 2 wheelers pose a higher risk of traffic accidents than do 4 wheelers, they are also considered to deliver various advantages regarding the protection of the global environment and the handling of traffic congestion. Their specific advantages include (1) CO₂ emissions from a vehicle is smaller, (2) speed of travel on congested road is faster, (3) impact on traffic capacity during actual road is smaller (Japan Automobile Manufacturers Association Inc 2009). However, it is expected that the overwhelming share accounted for by 2 wheelers in these Asian countries will come to be occupied by 4 wheelers as these countries experience further economic growth. It is therefore possible that the overall global carbon dioxide emissions from road traffic may increase greatly.

Based on this background, the authors conducted a NILIM-IRE joint research to elucidate the advantages (i.e., global environmental load reduction effects) of road traffic composed of 2 wheelers; road traffic that is composed of 2 wheelers is friendly to the global environment.

This paper shows the result of NILIM-IRE joint research, which is estimation of total CO₂ emission of road transport according to several scenarios of 2 wheelers ratio.

Figure 1. Road traffic modes in different countries (International Road Federation 2010)
2. TRIAL CALCULATION CONDITIONS AND PROCEDURE

Indonesia was selected as representative of countries in Asian countries with widespread 2 wheelers ownership. Japan was selected as representative of countries with scant 2 wheelers ownership. Road traffic modes of 2 wheelers and 4 wheelers were subjected to trial calculations. Buses and trucks were excluded from analysis. Figure 2 shows the trial calculation procedure. To conduct trial calculations of CO$_2$ emissions and vehicle kilometers traveled per year, we used formulae that are presented as follows.

\[
\text{CO}_2 \text{ emissions} = \text{Emission Factor } \times \text{Vehicle kilometers traveled per year} \quad (1)
\]

\[
\text{Vehicle kilometers traveled per year} = \text{Mileage per vehicle per year } \times \text{Number of vehicles owned} \quad (2)
\]

Scenarios for the conversion of the road traffic mode from 4 wheelers to 2 wheelers are the scenarios which are near to traffic mode in Japan or Indonesia, the scenario which supposed that the ratio of 2 wheeler is the same as 4 wheeler, the scenario which supposed that the ratio of 2 wheeler is not the same as 4 wheeler (see Table 1).

![Diagram of trial calculation procedure]

**Table 1. Scenarios describing the conversion of the road traffic mode from 4 wheelers to 2 wheelers**

<table>
<thead>
<tr>
<th>Scenario</th>
<th>4 wheelers</th>
<th>2 wheelers</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scenario 1</td>
<td>100%</td>
<td>0%</td>
<td>Close to the road traffic mode in Japan</td>
</tr>
<tr>
<td>Scenario 2</td>
<td>80%</td>
<td>20%</td>
<td></td>
</tr>
<tr>
<td>Scenario 3</td>
<td>50%</td>
<td>50%</td>
<td></td>
</tr>
<tr>
<td>Scenario 4</td>
<td>20%</td>
<td>80%</td>
<td>Close to the road traffic mode in Indonesia</td>
</tr>
<tr>
<td>Scenario 5</td>
<td>0%</td>
<td>100%</td>
<td></td>
</tr>
</tbody>
</table>

3. TRIAL CALCULATION

3.1 Number of vehicles owned

4 wheelers account for more than 90% of the number of vehicles owned in Japan, indicating that the 2 wheelers ownership rate is extremely small. By contrast, 2 wheelers account for more than 60% of the number of vehicles owned in Indonesia. Thus the 2 wheeler ownership rate in Indonesia is comparable to that in other Asian countries (see Figure 3). The numbers of vehicles owned in the above five scenarios are shown in Figure 4.
3.2 Mileage per vehicle per year

Mileage per vehicle per year was calculated by dividing the vehicle kilometers traveled per year by the number of vehicles owned. Because vehicle kilometers traveled in Indonesia was unknown, the average value for Asian countries was adopted. It was supposed that the mileage per vehicle per year before conversion would continue even with changes in the types of vehicle owned (see Figure 5). The vehicle kilometers traveled per year in the scenarios is shown in Figure 6.
3.3 Changes in traveling speed resulting from the conversion of the road traffic mode

With the conversion from 4 wheelers to 2 wheelers, the road space originally occupied by 4 wheelers is replaced by that occupied by 2 wheelers; thus the road occupation area of the vehicles decreases. Consequently, it was expected that average travel speeds would increase because the traffic jam of the road decreased. By contrast, it was expected that average travel speeds would decrease when 2 wheelers were replaced by 4 wheelers. Therefore, trial calculations were conducted in regard to changes in travel speeds based on the conversion of the road traffic mode as follows (see Table 2):

(1) Because travel speed data in Indonesia were unavailable, traffic volume survey data from the 2010 Road Traffic Census in Japan were used.
(2) By classifying travel speeds into six ranks of 10-km/h units, percentages of vehicle kilometers traveled in each rank were calculated based on 2010 Road Traffic Census data.
(3) Sections where average travel speeds close to the representative value of each rank (8, 20, 35, 45, or 55 km/h) were extracted from the 2010 Road Traffic Census data.
(4) The 2 wheelers ownership rate in Japan is closest to Scenario 1. Therefore, the average travel speed extracted in iii) was used as the average travel speed in Scenario 1.
(5) Based on road traffic theory (Japan Road Association 1984) in Japan, simulations were conducted on the sections to calculate changes in travel speed with the conversion of road traffic mode from 4 wheelers to 2 wheelers. However, it was supposed that for the rank 60 km/h and higher, no change in speeds would result from the conversion of the road traffic mode.

Table 2. Results of trial calculations on average travel speeds following the conversion of the road traffic mode

<table>
<thead>
<tr>
<th>Current travel speed class (km/h)</th>
<th>Scenario 1</th>
<th>Scenario 2</th>
<th>Scenario 3</th>
<th>Scenario 4</th>
<th>Scenario 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 wheelers 0%</td>
<td>8.9</td>
<td>15.6</td>
<td>22.1</td>
<td>26.9</td>
<td>30.1</td>
</tr>
<tr>
<td>2 wheelers 20%</td>
<td>18.5</td>
<td>20.2</td>
<td>23.3</td>
<td>26.7</td>
<td>29.0</td>
</tr>
<tr>
<td>2 wheelers 50%</td>
<td>32.7</td>
<td>38.2</td>
<td>39.2</td>
<td>40.2</td>
<td>40.9</td>
</tr>
<tr>
<td>2 wheelers 80%</td>
<td>47.3</td>
<td>48.0</td>
<td>49.2</td>
<td>50.3</td>
<td>51.1</td>
</tr>
<tr>
<td>2 wheelers 100%</td>
<td>50.1</td>
<td>50.7</td>
<td>51.6</td>
<td>52.5</td>
<td>53.1</td>
</tr>
</tbody>
</table>

3.4 CO₂ emissions factors by vehicle type

Based on the relationship between 4 wheeler travel speeds and CO₂ emissions, CO₂ emissions corresponding to average travel speeds for 4 wheelers following conversion of the road traffic mode were first calculated. Subsequently, by taking the weighted average of CO₂ emissions factors based on the vehicle kilometers traveled ratio in the Road Traffic Census, CO₂ emissions factors for 4 wheelers were obtained (see Figure 7 and Table 3).
Regarding 2 wheelers, CO₂ emissions corresponding to average travel speeds were first calculated based on the relationship between 2 wheeler travel speeds and CO₂ emissions. Subsequently, by taking the weighted average of CO₂ emissions factors based on the vehicle kilometers traveled ratio in the Road Traffic Census, CO₂ emissions factors for 2 wheelers were obtained (see Figure 8 and Table 4).

![Figure 7. Relationship between average travel speeds for 4 wheelers (passenger vehicles) and CO₂ emissions](image)

<table>
<thead>
<tr>
<th>Current travel speed class (km/h)</th>
<th>Vehicle kilometers traveled ratio (%)</th>
<th>Average travel speed (km/h)</th>
<th>Emissions factor (gCO₂/km)</th>
<th>Emissions factor (gCO₂/km)</th>
<th>Emissions factor (gCO₂/km)</th>
<th>Emissions factor (gCO₂/km)</th>
<th>Emissions factor (gCO₂/km)</th>
<th>Emissions factor (gCO₂/km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 10</td>
<td>0.1</td>
<td>8.9</td>
<td>373.9</td>
<td>15.6</td>
<td>253.6</td>
<td>22.1</td>
<td>210.5</td>
<td>26.9</td>
</tr>
<tr>
<td>10 - 30</td>
<td>25.5</td>
<td>18.4</td>
<td>232.8</td>
<td>20.2</td>
<td>220.0</td>
<td>23.3</td>
<td>204.5</td>
<td>26.7</td>
</tr>
<tr>
<td>30 - 40</td>
<td>21.9</td>
<td>32.7</td>
<td>169.4</td>
<td>38.2</td>
<td>156.0</td>
<td>39.2</td>
<td>153.8</td>
<td>40.2</td>
</tr>
<tr>
<td>40 - 50</td>
<td>20.7</td>
<td>47.3</td>
<td>141.8</td>
<td>48.0</td>
<td>140.8</td>
<td>49.2</td>
<td>139.1</td>
<td>50.3</td>
</tr>
<tr>
<td>50 - 60</td>
<td>11.7</td>
<td>50.1</td>
<td>137.9</td>
<td>50.7</td>
<td>137.4</td>
<td>51.6</td>
<td>136.7</td>
<td>52.5</td>
</tr>
<tr>
<td>60 -</td>
<td>20.0</td>
<td>60.0</td>
<td>130.0</td>
<td>60.0</td>
<td>130.0</td>
<td>60.0</td>
<td>130.0</td>
<td>60.0</td>
</tr>
<tr>
<td>Weighted average</td>
<td>-</td>
<td>168.6</td>
<td>-</td>
<td>162.0</td>
<td>-</td>
<td>157.0</td>
<td>-</td>
<td>152.4</td>
</tr>
</tbody>
</table>

![Figure 8. Relationship between travel speeds for 2 wheelers and CO₂ emissions](image)

Table 3 CO₂ emissions factors for 4 wheelers in the various scenarios

![Figure 8. Relationship between travel speeds for 2 wheelers and CO₂ emissions](image)

Figure 8. Relationship between travel speeds for 2 wheelers and CO₂ emissions (Manabu DOHI and Agah M. MULYADI 2013)
4. RESULTS OF TRIAL CALCULATION

4.1 Baseline CO₂ emissions

Baseline CO₂ emissions were calculated by multiplying the CO₂ emissions factors in the various scenarios, which are shown in Tables 3 and 4, by the vehicle kilometers traveled per year in the various scenarios, which are shown in Figures 6. The CO₂ emission factor for the scenario closest to the number of vehicles in each country was used as the baseline CO₂ emission factor. For Indonesia, Scenario 4 was closest, and for Japan, Scenario 1 was closest.

4.2 CO₂ emissions with the conversion from 4 wheelers to 2 wheelers

In Indonesia, the CO₂ reduction resulting from the conversion from 4 wheelers to 2 wheelers was calculated to be approximately 21 million tons by using the baseline as the reference. Meanwhile, the CO₂ increase caused by the conversion from 2 wheelers to 4 wheelers was calculated to be approximately 46 million tons. In countries that have higher ownership rates of 2 wheelers compared with 4 wheelers, it is expected that the conversion from 2 wheelers to 4 wheelers lead to a dramatic increase in CO₂ emissions, whereas the CO₂ reduction effect resulting from the conversion from 4 wheelers to 2 wheelers is small (see Figure 9).

In Japan, the CO₂ reduction resulting from the conversion from 4 wheelers to 2 wheelers was calculated to be approximately 66 million tons by using the baseline as the reference. Meanwhile, the CO₂ increase caused by the conversion from 2 wheelers to 4 wheelers was calculated to be approximately 1 million tons. In countries that have higher ownership rates of 4 wheelers compared with 2 wheelers, it is expected that the conversion from 4 wheelers to 2 wheelers lead to a dramatic reduction in CO₂ emissions, although the increase in CO₂ emissions resulting from the conversion from 2 wheelers to 4 wheelers is small (see Figure 9). However, the possibility of the conversion to 2 wheelers ownership in countries that have higher ownership rates of 4 wheelers compared with 2 wheelers is low; thus, it is considered that conversion to next-generation 4 wheelers will be more effective in CO₂ reduction in these countries.

---

### Table 4 CO₂ emissions factors for 2 wheelers in the various scenarios

<table>
<thead>
<tr>
<th>Current travel speed class (km/h)</th>
<th>Vehicle kilometers traveled ratio (%)</th>
<th>Average travel speed (km/h)</th>
<th>Emissions factor (gCO₂/km)</th>
<th>Average travel speed (km/h)</th>
<th>Emissions factor (gCO₂/km)</th>
<th>Average travel speed (km/h)</th>
<th>Emissions factor (gCO₂/km)</th>
<th>Average travel speed (km/h)</th>
<th>Emissions factor (gCO₂/km)</th>
<th>Average travel speed (km/h)</th>
<th>Emissions factor (gCO₂/km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 10</td>
<td>0.1</td>
<td>8.9</td>
<td>109.2</td>
<td>15.6</td>
<td>64.3</td>
<td>22.1</td>
<td>52.1</td>
<td>26.9</td>
<td>47.6</td>
<td>30.1</td>
<td>49.3</td>
</tr>
<tr>
<td>10 - 30</td>
<td>25.5</td>
<td>18.4</td>
<td>56.0</td>
<td>20.2</td>
<td>55.7</td>
<td>23.3</td>
<td>50.7</td>
<td>26.7</td>
<td>47.8</td>
<td>29.0</td>
<td>46.1</td>
</tr>
<tr>
<td>30 - 40</td>
<td>21.9</td>
<td>32.7</td>
<td>42.5</td>
<td>38.2</td>
<td>42.7</td>
<td>39.2</td>
<td>42.2</td>
<td>40.2</td>
<td>41.6</td>
<td>40.9</td>
<td>41.2</td>
</tr>
<tr>
<td>40 - 50</td>
<td>20.7</td>
<td>47.3</td>
<td>38.7</td>
<td>48.0</td>
<td>38.2</td>
<td>49.2</td>
<td>37.8</td>
<td>50.3</td>
<td>37.4</td>
<td>51.1</td>
<td>37.1</td>
</tr>
<tr>
<td>50 - 60</td>
<td>11.7</td>
<td>50.1</td>
<td>36.5</td>
<td>50.7</td>
<td>37.2</td>
<td>51.6</td>
<td>36.9</td>
<td>52.5</td>
<td>36.5</td>
<td>53.1</td>
<td>36.4</td>
</tr>
<tr>
<td>60 -</td>
<td>20.0</td>
<td>60.0</td>
<td>34.9</td>
<td>60.0</td>
<td>34.9</td>
<td>60.0</td>
<td>34.9</td>
<td>60.0</td>
<td>34.9</td>
<td>60.0</td>
<td>34.9</td>
</tr>
<tr>
<td>Weighted average</td>
<td>-</td>
<td>43.0</td>
<td>-</td>
<td>42.9</td>
<td>-</td>
<td>41.4</td>
<td>-</td>
<td>40.4</td>
<td>-</td>
<td>39.8</td>
<td></td>
</tr>
</tbody>
</table>

**Figure 9. Results of trial calculations**
5. CONCLUSION

In this study, we conducted trial calculations on CO₂ emissions with the conversion of road traffic mode from 4 wheelers to 2 wheelers in Asia in order to verify that 2 wheelers have advantages over 4 wheelers in road traffic and that road traffic composed of 2 wheelers is friendly to the global environment. The findings from the results of these trial calculations are summarized below:

(1) In countries that have higher ownership rates of 2 wheelers compared with 4 wheelers, it is expected that the conversion from 2 wheelers to 4 wheelers lead to a dramatic increase in CO₂ emissions, whereas the CO₂ reduction effect resulting from the conversion from 4 wheelers to 2 wheelers is small.

(2) In countries that have higher ownership rates of 4 wheelers compared with 2 wheelers, it is expected that the conversion from 4 wheelers to 2 wheelers lead to a dramatic reduction in CO₂ emissions, although the increase in CO₂ emissions resulting from the conversion from 2 wheelers to 4 wheelers is small. However, the possibility of the conversion to 2 wheelers ownership in countries that have higher ownership rates of 4 wheelers compared with 2 wheelers is low; thus, it is considered that conversion to next-generation 4 wheelers will be more effective in CO₂ reduction in these countries.

It is indicated that road traffic composed of 2 wheelers is friendly to the global environment. However, 2 wheelers are not suited to inter-city transportation such as logistics, though they are suited to relatively short trips such as commuting within a city. Therefore, it is necessary to examine for what kinds of trip the conversion from 2 wheelers to 4 wheelers should be controlled. Because 2 wheelers pose a higher risk of traffic accidents than do 4 wheelers, it is necessary to examine road safety measures, which are done in Indonesia now, and 2 wheeler safety measures will be able to contribute CO₂ reduction in the future.

REFERENCES
National Institute for Land and Infrastructure Management. (2003). Calculation of emission factor of CO2, NOx and SPM to use for the calculation of the quantitative index, Tsukuba.
Manabu DOHIL, and Agah M. MULYADI.(2013). Bilateral joint research on Environmentally Friendly Transport System, using Motorcycle, 14th REAAA Conference, REAAA.
A Study on the Improvement of Private-Funded Expressways Operated by a Public Corporation

**KEYWORDS:**
PPP, private-funded, private expressways, expressway operation system

**ABSTRACT:**
Although the Korean government controlled and managed all expressways throughout the country until the 1990s, it started to accept private investments for constructing expressways for the past decade. As of 2014, 464km of 10 private expressways are under private operation, and 14 are planned to be built additionally with private investments. Naturally, the private and public companies are competing with each other to provide better services.

This paper aims at developing an optimal operation model for public expressways by comparing the different operation method between private and public companies. The research result especially focuses on the reduction of operation cost and the efficiency of expressway operation system by studying the process of expressway operation, organization in charge of operation, human resources involved, and operation cost.

Since personnel expenses take the largest proportion among all cost-related items, personnel reduction is the key factor for reducing the operation cost. In addition, certain tasks required in the operation process should be outsourced to professional organizations to maintain cost-effective services, as long as it does not infringe public interest.

The improved operation and maintenance strategies for private-funded expressway will not only reduce operation costs, but eventually ensure safety and convenience of the drivers.
A Study on the Improvement of Private-Funded Expressways Operated by a Public Corporation

Hong Suk-kee, Choi Jong-chul, Lim Joonbeom, Lee Soobeom

University of Seoul, Seoul, Republic of Korea
Email for correspondence: jackhong7@gmail.com

1. INTRODUCTION

The construction and operation of expressways used to be publicly-funded business in most cases. However, starting with the opening of the New Airport Hiway in 2000, 10% of total expressways, 10 routes consisting of 464km of roads in total - are currently being financed by private capital. The operation of a total of 24 privately-funded expressways is planned. Out of 10 private expressways that are operated and managed by private entities, the Korea Expressway Corporation (KEC) is directly involved in the operation and maintenance of three routes: the Seoul-Chuncheon, the Busan-Ulsan, and the Seo (west) Suwon-Pyeongtaek, and has plans to manage an additional 11 private expressways following the integrated operation plan of the Ministry of Land, Transport and Maritime Affairs of Korea (MLTM) of the Republic of Korea.

However, those routes other than the three private routes directly managed by the KEC are on average less than 40km long, and inefficiencies result due to maintenance by a single business operator and the need for expressway users’ to frequently stop for toll collection. Problems arise with private expressways whose average length is as short as 39km, because of toll collection and the need for different maintenance processes for their respective route, consequently causing user inconvenience, road congestion, and the increase in the operation cost. The government, with an eye on resolving such problems, developed the “Measures for the Integrated Operation of Private Expressways (MLTM, 2010)” based on which the integrated operation of two to three short-distance private expressways may be possible, therefore ultimately saving in maintenance fees through economies of scale and the improvement of customer convenience through the establishment of an integrated toll system.

In time, the operation cost of running private expressways is becoming more competitive, and the maintenance cost is also decreasing. Thus, the need to analyze the operation models of private expressways to make improvements was raised with the KEC, which will play a pivotal role in private expressways’ maintenance according to the MLTM’s Measures for the Integrated Operation of Private Expressways. For the sake of the efficient operation of private expressways, it is important to make a comparative study with expressways already operated by existing private operators to find suitable improvement measures.

This study compares private expressways operated by the KEC and private operators in terms of operation and maintenance process, operation and maintenance organization, the number of office personnel, and the operation costs. Based on the findings, the study proposes measures to save on the operation costs and to improve efficiencies in the operation system for the 3 routes which are operated by KEC. First, this study examined the adjustment of toll gate office personnel to make savings on the operation costs. Regarding outsourcing candidates, this study suggests several alternatives on the premise that possible violation of public interests is sufficiently taken into consideration. Also, new measures are proposed for the increase of profitability to bring about more eco-friendliness and efficiencies in spaces utilizing road assets, such as integrated facilities in rest areas and road-side areas.

2. CURRENT STATUS OF PRIVATE EXPRESSWAYS

Private investments started in earnest after the enactment of the Promotion of Private Capital into Social Overhead Capital Investment Act in 1994. Projects promoted since the enactment of the Act can be divided into three phases. The 1st phase was between August 1994 when the Act came into effect and December 1998 when the Act was revised to the Act on Public-Private Partnerships in Infrastructure. The 2nd phase was between January 1999 after the revision of the Act and December 2005, prior to the abrogation of the Minimum Revenue Guarantee (MRG). The 3rd phase was between 2006, at the time of abrogation of the MRG, and the present. As shown in Table 1, a total of 10 private expressways are in operation as of 2014, and 10 projects are under construction or under negotiation. Furthermore, the Pocheon-Hwado, the Songsan-Bongdam, and the Icheon-Osan routes are under current negotiation, and the Gongjiu-Cheongwon route is being proposed. A total of 24 private expressways are expected to be in operation by 2018.
Starting with the Incheon International Airport Expressway, private expressways started opening in the 2000s, such as the Cheonan-Nonsan, and the Daegu-Busan routes. The KEC, with an aim to secure business and improve its public perception, took charge of operating the Busan-Ulsan Highway in 2008, and the Seoul-Chuncheon Highway and the Seo Suwon-Pyeongtaek Highway in 2009 from private operators. The current status is shown in Table 2. The KEC plans to operate an additional 11 routes (excluding those in usage shown in Table 1, and the Gwangju-Wonju, the Songsan-Bongdam, the Seoul-Munsan Highways) according to the Measures for the Integrated Operation of Private Expressways (MLTM, 2010).

Table 1. Length of private expressways (as of 2014)

<table>
<thead>
<tr>
<th>Status</th>
<th>Route</th>
<th>Length (km)</th>
<th>Status</th>
<th>Route</th>
<th>Length (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>In usage</td>
<td>Incheon Intl. Airport</td>
<td>40.2</td>
<td>Under construction</td>
<td>Suwon-Gwangmyeong</td>
<td>27.4</td>
</tr>
<tr>
<td></td>
<td>Cheonan-Nonsan</td>
<td>81.0</td>
<td></td>
<td>Gwangju-Wonju</td>
<td>57.0</td>
</tr>
<tr>
<td></td>
<td>Daegu-Busan</td>
<td>82.0</td>
<td></td>
<td>Incheon-Gimpo</td>
<td>28.5</td>
</tr>
<tr>
<td></td>
<td>Ilsan-Toegyewon</td>
<td>36.3</td>
<td></td>
<td>Anyang-Seongnam</td>
<td>21.9</td>
</tr>
<tr>
<td></td>
<td>Busan-Ulsan</td>
<td>47.2</td>
<td></td>
<td>Sangju-Yeongcheon</td>
<td>93.9</td>
</tr>
<tr>
<td></td>
<td>Seoul-Chuncheon</td>
<td>61.4</td>
<td></td>
<td>Guri-Pocheon</td>
<td>50.5</td>
</tr>
<tr>
<td></td>
<td>Seo Suwon-Pyeongtaek</td>
<td>38.5</td>
<td></td>
<td>Busan new port 2nd back road</td>
<td>15.3</td>
</tr>
<tr>
<td></td>
<td>Incheon Bridge</td>
<td>12.3</td>
<td></td>
<td>Oksan-Ochang</td>
<td>12.1</td>
</tr>
<tr>
<td></td>
<td>Yongin-Seoul</td>
<td>22.9</td>
<td>Under negotiation</td>
<td>Gwangmyeong-Seoul</td>
<td>20.0</td>
</tr>
<tr>
<td></td>
<td>Pyeongtaek-Siheung</td>
<td>42.6</td>
<td></td>
<td>Seoul-Munsan</td>
<td>35.6</td>
</tr>
</tbody>
</table>

* Total length of above 20 expressways: 826.6km (41.3km on average)

Table 2. Private expressways operated and managed by KEC (as of 2014)

<table>
<thead>
<tr>
<th>Operator</th>
<th>Sum</th>
<th>Busan-Ulsan</th>
<th>Seoul-Chuncheon</th>
<th>Seo Suwon-Pyeongtaek</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (km)</td>
<td>147.1</td>
<td>47.2</td>
<td>61.4</td>
<td>38.5</td>
</tr>
<tr>
<td>Operation period (yrs)</td>
<td>-</td>
<td>30</td>
<td>30</td>
<td>30</td>
</tr>
</tbody>
</table>

3. ESTABLISHMENT AND OPERATION OF THE PRIVATE EXPRESSWAY INTEGRATED SYSTEM

The method for performing integrated management is well explained in the Measures for the Integrated Operation of Private Expressways (MLTM, 2010). For the 10 private expressways currently in operation, there are 20 toll gates altogether to collect the toll fee for each route. When drivers take the private-funded expressways “Seo Suwon-Pyeongtaek → Suwon-Gwangmyeong route → Gwangmyeong-Seoul,” they need to stop six times and pay the toll three times. Meanwhile, when they drive through the KEC-operated expressways “Gyeongbu line → Cheonan-Nonsan (Private) → Honam line,” they only need to stop four times and pay the toll three times.

Although the average length of the 20 private expressways is only 41km, drivers have to stop frequently for paying the tolls because different private operators have separate toll and maintenance systems for their expressways. Consequently, a necessity for the integrated operation of nearby private expressways has emerged, in order to resolve user inconvenience and decrease the operation costs. Thus, this study looked into the integrated toll collection and maintenance systems that are currently in use.

Toll Collection

For mutually connected private expressways which are possible to integrate without investing additional costs, the toll collection is integrated as shown in Table 3 in order to minimize the number of stops and tollgate installation. As a mid-to-long term goal, the toll collection system is being integrated for public expressways operated by the KEC, and after the integration, KEC collects the toll and distributes it to each private operator. The cost may be saved for some routes by reducing the number of tollgates.
Table 3. Integrated toll collection system for private-invested expressways (MLTM, 2010)

<table>
<thead>
<tr>
<th>Routes</th>
<th>Length (km)</th>
<th>Number of toll gates</th>
</tr>
</thead>
<tbody>
<tr>
<td>Toll collection + Maintenance</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seo Suwon-Pyeongtaek + Suwon-Gwangmyeong + Gwangmyeong-Seoul</td>
<td>84.7</td>
<td>4</td>
</tr>
<tr>
<td>Guri-Pocheon + Pocheon-Hwado</td>
<td>77.0</td>
<td>1</td>
</tr>
<tr>
<td>Pyeongtaek-Siheung + Songsan-Bongdam</td>
<td>61.1</td>
<td>1</td>
</tr>
<tr>
<td>Only Maintenance</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seoul-Chuncheon + Hwado-Yangpyeong</td>
<td>80.0</td>
<td>-</td>
</tr>
<tr>
<td>Ilsan-Toegyewon + Seoul-Munsan</td>
<td>71.0</td>
<td>-</td>
</tr>
</tbody>
</table>

Operation and Maintenance

Private expressways, whose average length is currently only about 40km, should be extended to 60 - 90km by integrating the neighbouring private and public expressways. In this way, operation costs can be saved by integrating the operation and maintenance systems, facilities and equipment, and jointly outsourcing common needs.

Table 4. Operation method for integrated government-invested and private-invested expressways (MLTM 2010)

<table>
<thead>
<tr>
<th>Integrated routes</th>
<th>Length (km)</th>
<th>Regional Office in Charge</th>
</tr>
</thead>
<tbody>
<tr>
<td>Busan-Ulsan (47.2) + Gyeongbu Exp (29.4) + Ulsan Exp (14.3)</td>
<td>90.9</td>
<td>Ulsan Office</td>
</tr>
<tr>
<td>Oksan- Osan (12.1) + Gyeongbu Exp (55.5)</td>
<td>67.6</td>
<td>Cheonan Office</td>
</tr>
<tr>
<td>Osan-Gwangju (29.7) + Gyeongbu Exp (55.6)</td>
<td>85.3</td>
<td>Suwon Office</td>
</tr>
<tr>
<td>Gongju-Cheongwon (20.1) + Daejeon-Dangjin (78.6)</td>
<td>98.7</td>
<td>Gongju Office</td>
</tr>
<tr>
<td>Anyang-Seongnam (21.9) + 2nd Gyeongin (16.9) + Seohaean line (14.6) + Seoul Ring Road (19.9)</td>
<td>73.2</td>
<td>Siheung Office</td>
</tr>
<tr>
<td>Incheon-Gimpo (28.5) + Gyeongin (23.9) + Siheung-Ilsan (24.8)</td>
<td>77.2</td>
<td>Incheon Office</td>
</tr>
<tr>
<td>Busan New port 2nd back road (15.3) + Namhae line (80.1)</td>
<td>95.4</td>
<td>Changwon Office</td>
</tr>
</tbody>
</table>

4. OPERATION MODELS OF EXPRESSWAYS OPERATED BY KEC AND PRIVATE COMPANIES

Selecting the Target for Comparing the Operation Models

When comparing the length of expressways operated by the KEC and private operators, their distance and characteristics must be similar in order to guarantee credibility. As explained earlier, the KEC is responsible for the maintenance of three routes (Seoul-Chuncheon, Seo Suwon-Pyeongtaek, Busan-Ulsan), and the comparison targets of private operators are seven routes, among which three routes, the Cheonan-Nonsan, the Daegu-Busan, and the Seoul Ring Road (Ilsan-Toegyewon), appear to be similar in terms of characteristics and length with the three routes operated by the KEC. They were selected to be comparison targets as shown in Table 5.

Other expressways that have shorter lengths or different characteristics were excluded from comparison. The Yongin-Seoul Expressway was excluded because of its short length of 31km, and the Incheon Bridge and the New Airport Hiway routes were also excluded because they exist for the three bridges, Incheon, Yeongjong Grand, and Banghwadaegyo, and thus, have different characteristics from private expressways managed by the KEC. As a result, the comparison targets were limited to the Cheonan-Nonsan, the Daegu-Busan, and the Seoul Ring Road. Also, the Pyeongtaek-Siheung Expressway was excluded as it was newly opened in 2013 and as yet lacks sufficient data.

Table 5. Private expressways for comparison with expressways managed by KEC (KEC 2012)

<table>
<thead>
<tr>
<th></th>
<th>Average</th>
<th>Cheonan-Nonsan</th>
<th>Daegu-Busan</th>
<th>Seoul Ring Road</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (km)</td>
<td>47.3</td>
<td>80.96</td>
<td>82.05</td>
<td>36.30</td>
</tr>
<tr>
<td>Converted Length (km)</td>
<td>56.2</td>
<td>81.0</td>
<td>82.1</td>
<td>58.6</td>
</tr>
<tr>
<td>Serviced Since</td>
<td>-</td>
<td>Dec 2002</td>
<td>Jan 2006</td>
<td>Dec 2007</td>
</tr>
</tbody>
</table>
Comparison of Operation and Maintenance Models

Apparently, the overall work process of the expressways managed by the KEC and the three expressways managed by private operators are similar. With regards to the organizational structure, the expressway directly operated by private operators consists of Management Support Team, Operation Support Team, Road Maintenance Team, Traffic Support Team, and Customer Support Team as shown in Figure 1. The works based on the organizational structure are similar to the KEC. However, the outsourcing process shows slight differences as shown in Table 6.

Figure 1. Organization for managing private expressways and KEC

Table 6. Work process method of KEC and private expressways (KEC, 2012)

<table>
<thead>
<tr>
<th></th>
<th>KEC</th>
<th>Cheonan-Nonsan</th>
<th>Daegu-Busan</th>
<th>Seoul Ring Road</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>General maintenance</strong></td>
<td>partly outsourced</td>
<td>directly managed</td>
<td>outsourced</td>
<td>outsourced</td>
</tr>
<tr>
<td><strong>Traffic monitor</strong></td>
<td>directly managed</td>
<td>outsourced</td>
<td>outsourced</td>
<td>outsourced</td>
</tr>
<tr>
<td><strong>Structure inspection</strong></td>
<td>directly managed</td>
<td>outsourced</td>
<td>outsourced</td>
<td>outsourced</td>
</tr>
<tr>
<td><strong>Regular patrol</strong></td>
<td>outsourced</td>
<td>outsourced</td>
<td>outsourced</td>
<td>outsourced</td>
</tr>
<tr>
<td><strong>Repair &amp; maintenance</strong></td>
<td>outsourced</td>
<td>outsourced</td>
<td>outsourced</td>
<td>outsourced</td>
</tr>
</tbody>
</table>

As shown in Table 6, the KEC directly manages general maintenance, traffic monitoring and structure inspection which may directly affect expressway users. It outsources regular patrol and repair & maintenance. To save operation costs, private operators outsource most works except for administrative tasks. However, the Cheonan-Nonsan Expressway takes charge of general maintenance because manpower supply is limited in the region.

The number of personnel working for the KEC and for the three private expressways according to the "Establishment of Standard Model for Management Office of Private Expressways (KEC, 2012)” is shown in Figure 2. The total number of personnel per 10km at the KEC is approximately 1.3 higher than for private operators. For each department, the KEC has more management (0.1 person) and operation (1.7 person) personnel, and fewer road/structure (0.2 person) and traffic (0.4 person) personnel. Three departments, except operation, show that the KEC and the three private expressways have a similar number of personnel, or the KEC has fewer personnel. This is because the KEC integrated its systems with public highways except for the operation division. To save the general operation costs, the KEC must reduce the number of personnel in operation division.
Figure 2. Number of staffs at KEC and private company (KEC, 2012)

Regarding the operation costs, general maintenance costs consist of personnel expenses and maintenance costs. Personnel expenses include payroll costs and other employee benefits of personnel necessary for operation and maintenance. Maintenance costs include those costs used to manage expressways and reinvestment costs used to replace facilities.

The Establishment of a Standard Model for the Maintenance Office of Private Expressways (KEC, 2012) compared the operation costs of the KEC and three private expressways of private operators, and the results are as shown in Figure 3. The operation cost per km is KRW 214 million for the KEC and KRW 244 million for private operators. Private operators spend approximately KRW 30 million per km more than the KEC. The personnel expenses are KRW 38 million for the KEC and KRW 32 million for private operators, which means the KEC spends approximately KRW 6 million per km more. The maintenance costs are KRW 176 million for the KEC and KRW 212 million for private operators, which means the private operators’ costs are approximately KRW 36 million more than the KEC.

Figure 3. Operation cost at KEC and private company (KEC, 2012)
5. MEASURES TO IMPROVE THE MAINTENANCE OPERATION MODELS OF PRIVATE EXPRESSWAYS

As can be seen from the above examination, there is no significant difference in the maintenance work process and organizational structures of private expressways. There is some difference in terms of outsourcing, but this is due to the difference in the characteristics of public interest-pursuing KEC and profit-seeking private operators. In order to improve cost-efficiency, the KEC may consider the outsourcing of some areas that will not hinder the security of the public interest. However, the KEC is a public organization following government legislation, and thus it cannot bid and sign contracts at competitive prices compared to private operators. It may cause difficulties in ensuring cost-efficiency from outsourcing like private operators.

Based on the comparison of operation models, this study proposes improvement measures including the effective management of personnel at toll gate office, expanding outsourcing, and increasing profitability.

Effective Management of Personnel at Toll Gate Office

The KEC is more efficient in the organization and management of personnel compared to private operators, except with regard to operation, in which area it must seek improvement. The KEC has a comparatively higher number of office management personnel for the private expressways which it operates compared to private operators. The number of personnel per 10km did not incorporate the number of toll gate offices, therefore making it difficult for direct comparison. When the number of management personnel was recalculated based on the number of toll gate offices, there were approximately 2 times more personnel for expressways managed by KEC compared to expressways managed by private operators, as can be seen in Table 7.

Table 7. Number of staff per toll gate office

<table>
<thead>
<tr>
<th></th>
<th>Expressways managed by private operators</th>
<th>Expressways managed by KEC</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of staff per office</td>
<td>0.9</td>
<td>1.9</td>
<td>2.1 times</td>
</tr>
<tr>
<td>Management Personnel</td>
<td>20</td>
<td>44</td>
<td></td>
</tr>
<tr>
<td>No. of toll gate offices</td>
<td>23</td>
<td>23</td>
<td>–</td>
</tr>
</tbody>
</table>

It may be concluded that the KEC's operation costs are lower, thus more efficient, than other private operators. However, since personnel cost is higher than other private operators, improvement measures must be found and applied for operation as mentioned before. The KEC plans to establish a staged personnel reduction plan and reduce the number of its personnel from 2 persons to 1 person per toll gate office, which will consequently enhance efficiency in terms of personnel management and cost reduction. Furthermore, the KEC must reduce the number of management personnel from 1 to 0.5 person by integrating two toll gate offices, so that the number will be adjusted to the level of other private operators.

Expansion of Outsourcing

Taking account of private expressways operated by private operators, it is possible for the KEC to partially outsource works that are directly performed by the corporation on private expressways. It is also judged that it is appropriate to initially undertake a pilot project and expand it in the long term. Works that may be eligible for outsourcing are patrolling, traffic monitoring, and general maintenance, as shown in cases of private expressways of private operators. However, as described in Figure 4, any possible violation of public interests must be sufficiently considered before deciding on the applicability.
### Problems caused by outsourcing

<table>
<thead>
<tr>
<th>Regular patrolling</th>
<th>Traffic monitoring</th>
<th>General maintenance</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Inappropriate measures may be taken in case of emergency due to low sense of responsibility (which eventually violates public interest)</td>
<td>• No noticeable effect of outsourcing in both financial and non-financial aspect</td>
<td>• Staffs do not feel appreciated as a member of the organization, as only simple tasks are assigned</td>
</tr>
<tr>
<td>• No noticeable effect of outsourcing in short term</td>
<td></td>
<td>• Inappropriate measures may be taken in case of emergency due to low sense of responsibility (which eventually violates public interest)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Actions (e.g. termination of contract) are required for those who show low performance or violate public interest</td>
</tr>
<tr>
<td>• Will be cost-effective in the long run</td>
</tr>
<tr>
<td>• Work experience as a secure patroller must be required for the traffic monitoring position</td>
</tr>
<tr>
<td>• Traffic monitoring and secure patrolling tasks must be connected to one another, as secure patrolling is mostly outsourced nowadays</td>
</tr>
<tr>
<td>• Actions (e.g. termination of contract) are required for those who show low performance or violate public interest</td>
</tr>
<tr>
<td>• Hardly any private operators violated public interest so far</td>
</tr>
</tbody>
</table>

### Increase of Profitability and Saving on Costs

The major purpose of expressways is to provide transport routes to vehicles, but profits can also be made by utilizing expressway assets. Currently, expressways are making profits with their side businesses, such as rest areas and gas stations, and are able to explore additional profit models in diverse areas, which to some degree will offset the operation costs. The KEC has applied or will apply several side business models, such as developing abandoned roads, and a whole new concept of integrated facilities at rest areas to replace the old type of resting facilities. If automated cameras are used for toll collection, significant costs can be saved on toll collectors or booth installation sites which will no longer be needed. Efforts must be made to discover ways to create additional profits and save on costs by using IT. When contracting with private operators and including articles regarding an increase in profits and cost saving, such conditions must be explicitly stated in the agreements or negotiated with private operators so that they will not affect the operation costs of the KEC.

### 6. CONCLUSION

A total of 10 privately funded expressways, which started to be promoted in 1995, are currently in operation, and the number will increase to 24. However, problems such as user inconvenience, congestion, and increase in the operation costs are on the rise as private expressways collect tolls and perform maintenance for each route within the short average length of mere 39km. The government, with the aim of resolving such problems, established the Measures for the Integrated Operation of Private Expressways (MLTM, 2010), and promoted the integrated operation of two or three short-distance private expressways so as to save the maintenance costs through economies of scale and the improve customer convenience through the establishment of an integrated toll collection system.

This study, for the purpose of resolving road user inconvenience, making budget savings, and expanding businesses and securing public interest for integrated operation on the national level, concluded that the KEC must make efficiency improvements in maintenance of its soon-to-be-managed 11 private expressways. Thus, this study compared and analyzed case studies of private expressways in operation, and proposed the direction for the KEC to take when being entrusted in and operating additional private expressways in the near future.

The three private expressways currently operated by the KEC and other three private expressways with similar characteristics and lengths were compared in terms of the operation and personnel costs. The necessary steps that the KEC must take in order to save budgets of routes in service are as follows:

- The private expressways operated by the KEC have a higher proportion of personnel cost than private expressways directly operated by private operators. Therefore, for efficient operation, the KEC shall review and apply a reduction of toll gate office personnel.

- In order to save on maintenance costs, the KEC shall review the possibility of additional outsourcing as long as it does not infringe upon the public interests.

- In order to increase profitability and save on costs, the KEC shall find and apply diverse profit models, such as developing abandoned roads.
REFERENCES

1. Board of Audit and Inspection of Korea (2011), Cheonan-Nonsan, Daegu-Busan, Seoul Ring Road Expressway Audit Report
2. KEC (2011), Detailed regulation for the organization
4. KEC (2014), Expressway Work Statistics
7. Ministry of Land, Transport and Maritime Affairs of Korea (2011), Basic Plan for Road Modification II
9. Websites
   - Cheonan-Nonsan Expressway, http://www.cneway.co.kr
   - New Daegu-Busan Expressway, http://www.dbeway.co.kr
   - Seoul Expressway, http://www.seoulbeltway.co.kr

198
Establishing Optimal Long Term Funding Allocation Systematic Approach based on Network Needs & Availability of Funds

Alan Roland, Prof Mark Porter, Prof. John Yeaman,
The University of the Sunshine Coast, Queensland, Australia

Summary:
The research will provide a solid basis for a systematic approach and resulting a tool for assessing fund forecasts considering various scenarios. This system will be flexible and can be adjusted to provide a confidence to the road authorities and governments on needs verses available revenue. The system also will assist the government funding agency to compile and integrate funding request from various authorities to optimize fund bids.

This research will be unique in that it will provide the road network owner’s viewpoint and be driven by value for money rather than profit whilst meeting the fundamental principles of sustainability, innovation and risk management.

Keywords: Asset Management, Funding Allocation, Budgeting Framework, Allocation Mechanisms.

1 INTRODUCTION

Road Controlling Agencies (RCAs) around the world are facing continuous challenges sustaining network funds. Given the fluctuation of market forces and instability of economies around the world. Effective management of a road network requires that levels are set at least sufficient to keep the core of road assets in a stable condition for long term. This requires that ongoing maintenance is funded, and that adequate provision is made for any strengthening works required. More than this minimum level will be required if the network is to be expanded or improved. Roads are usually funded through budget allocations determined as part of the annual government budgeting process.

Roads with different hierarchies and functional classifications may be managed by different road administrations. Sometimes they will have their own sources of funds (investments). For others, funding will come from the national sources. Particularly for roads of lower hierarchy, the provision of funds will often be shared between national and local / regional sources. It is common for central governments to fund all work on national or trunk roads. In all cases, mechanisms need to be in place for allocating and disbursing funds between the different administrations. They need to be simple, transparent and encourage consistency of standards between the different administrations. Example case and a reference made for some of the elements described in this paper from Abu Dhabi as the researcher is currently working over there.

2 BUDGETING ADMINISTRATIVE FRAMEWORK

Modern budgeting systems were developed as a means of exerting legislative control over resource allocation decisions by the executive. This was achieved by dividing responsibility for and authority over the resource allocation process between institutions whose competencies and relations were defined in law, supplemented by exhaustive rules and procedures.

<table>
<thead>
<tr>
<th>Ministry of Finance</th>
<th>Responsible for the management of public expenditure, including the formulation of a consolidated state budget and accounts, and the management of government’s cash resources.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spending Agencies</td>
<td>Responsible for the planning, management and delivery of public services, and the preparation and management of agency budget. Spending agencies are usually headed Ministers, occasionally by public officials.</td>
</tr>
<tr>
<td>Cabinet</td>
<td>Collectively formulates government policy. Implementation of government policy is the responsibility of individual Ministers. Cabinet approves the Government’s budget.</td>
</tr>
<tr>
<td>Legislature</td>
<td>Analyses the Government’s budget proposal and accounts, through the work of specialist committees, and enacts the budget in law. In Congressional systems, the legislature may amend the Government’s budget proposal. In Parliamentary systems, it usually may not.</td>
</tr>
<tr>
<td>Auditor</td>
<td>Verifies compliance with the budget law and procedures regarding the use of public funds. The Auditor usually reports directly to the legislature, though in some cases may be considered part of the Ministry of Finance and report to Government through the Minister.</td>
</tr>
</tbody>
</table>

3 STRATEGIC BUDGETING

A lump sum budget is awarded and decisions on its expenditure between different types of works are made by the road administration itself on the basis of policy framework or needs. Local budget categories may be used to assist in managing funds. While a unified budget will enable the needs of the network to be considered as a whole, the possibility of optimizing
expenditures may not be optimal and could vary year by year; this research will assist to establish the necessary breakdown of budget categories, which can be summed for a unified lump sum.

4 LINKING BUDGETS TO BUSINESS OBJECTIVES

When the revenues available to the road sector are significantly less than the amount required to maintain the road network in a stable long-term condition and to undertake justified improvements, the road agency should prepare an explicit long-term financing plan showing the size of the financing gap and the options for bridging it. Having identified the size of the financing gap, if any, the financing plan should then consider ways of reducing it. First, it should consider the scope for increasing the revenues available to the road sector by simplifying road user taxes and charges, restructuring them and reducing avoidance, evasion and leakage. This research should assist to in closing the gap in the current / optimal practice and creating a systematic process in a wide range of scenarios.

5 IMPORTANCE OF RESEARCH

The research will provide a system tool for assessing fund forecasts considering various scenarios. This system will be flexible and can be adjusted to provide a confidence to the road authority and government on needs verses available revenue. The system also will assist the government funding agency to compile and integrate funding request from various authorities to optimize fund bids.

6 RESEARCH LITERATURE REVIEW

It is important to differentiate between financing and funding. The term funding, as used in this research, refers to how infrastructure is paid for. Ultimately, there are only two sources of funding for infrastructure, government investment or direct user charges. This is opposed to financing, which refers to the way in which debt and/or equity is raised for the delivery and operation of an infrastructure project.

The budget process has two principal components:

- To decide how much funds are needed;
- To decide how to allocate the funds those are actually awarded.

Usually, budgets are allocated on an annual basis, a typical budget process might be summarized as follows,

- There is an initial call from a Finance Authority or Ministry to submit bids;
- Spending authorities respond;
- Subsequently the Finance Authority consolidates and reviews the submissions within overall spending targets;
- Government authorities may be called to support their submission during this process;
- Draft estimates are then published and are submitted to elected members of the government for approval, (in the case of Abu Dhabi, the Executive Council) will be the approving authority;
- Following this, a warrant for a given amount is issued to relevant authorities, so that spending may commence at the start of the new financial year.

Under such arrangements, therefore each authority competes for funds and at least in theory, funds are allocated to finance those expenditures with highest economic or social return, however such allocation are invariably highly politicized and allocations are often far from economically optimal. Politicians at national level are more likely to reflect general social needs than at the local level, where vested interests tend to have greater influence. Expenditures for maintenance, in all sectors inevitably lose out to higher profile capital investment projects, which contribute to the under funding of roading maintenance, as a result roading maintenance expenditures are often based on historical precedent; each year’s budget is based on that of the previous year, with an additional allowance to cover inflation. This is a limited and poor basis for budgeting since it is arbitrary.

A better robust approach is for the budget application to be assessed on a rational assessment of economic need that relates to the objectives specified in the policy framework. One approach to the needs-based budgeting is for budgets to be based on life cycle costs, upgrading and reconstruction costs and roading users costs over the life of the road by choosing the optimal level of maintenance. If roads are maintained too soon, then the full value of the existing pavement will not be attained and maintenance costs will higher than any reduction in vehicle operating costs (VOC), hence, total transportation costs are higher; conversely, if roads are maintained too late, the consequent maintenance will be more expensive or the value of asset may be lost. Therefore, the standards and intervention levels specified in the policy framework should reflect the need for
maintenance. If there is to be consistent within the policy, the road maintenance budget, then the cost of the work needed to correct defects should be funded by the budget.

A review of the funding allocation process used in a number of countries around the world has been undertaken, such as New Zealand, Australia, North America, Europ, Asia and Africa, it important to note, while this review may discuss how the government funding agency distributing the fund, the reality compiling a bid by the authority reflecting how detailed or a sophisticated process being used by the funding authority.

7 OPTIMAL PRACTICE WORLD WIDE – SUMMARY REVIEW

For efficient and consistent allocation of monies, priorities should be on the basis of economic cost-benefit principles, selecting those projects that demonstrate the highest economic rates of return. In practice the task is not that simple. Political interests may impose certain regional allocations, and rural roads need to be justified on the basis of other criteria because their economic case will inevitably be weak. The process of allocating monies between types (trunk, district, urban and rural or feeder) is a continuing problem that has not been satisfactorily been resolved. Some Road Funds are committed to a fixed percentage allocation, but this is not necessarily the best solution, and cannot always be affected anyway.

Resource allocation decisions are made unilaterally by the Ministry of Finance. In many cases, however, allocations are made in consultation with the Ministry of Transport. The consultation process can assist the Ministry of Finance in determining the appropriate total level of funding to be made available for road and bridge improvements. This determination must consider a total maintenance and rehabilitation requirements to support the desired level of overall system condition and performance for the country road and bridge systems.

8 BUDGET CATEGORIES

Budget Categories, the financial provision of roading network in many countries is divided into:

Capital (Capex) which relate to the construction of new roads, and sometimes the reconstruction, rehabilitation, strengthening and rescaling or renewals of existing roads;

Recurrent (Operational) – the other category is a provision for the regular maintenance of the existing roading network, such as surfaces, off carriageway features, and for dealing with various contingencies, staff cost may also be paid under this category.

Sometimes the strengthening and renewals may be paid within the recurrent budget. In some roading agencies, the budget categories have breakdowns into more details such as a budget to maintain road structure, corridor maintenance, drainage facilities etc.

Where budgets awarded under different budget categories are usually less than bids for the roading agency, this means a lack of ability to vary funds from one budget category to another and will prevent the optimal allocation of resources under overall budget constraint.

9 ALLOCATION MECHANISMS

Roads with different hierarchies and functional classifications may be managed by different road administrations. Sometimes they will have their own sources of funds (investments). For others, funding will come from the national sources. Particularly for roads of lower hierarchy, the provision of funds will often be shared between national and local / regional sources. It is common for central governments to fund all work on national or trunk roads. In all cases, mechanisms need to be in place for allocating and disbursing funds between the different administrations. They need to be simple, transparent and encourage consistency of standards between the different administrations. Three basic methods are commonly used for:

- Simple Allocation Formula;
- Indirect Assessment of Needs;
- Direct Assessment of Needs.

Simple allocation formula assigns funds on the basis of pre-defined percentages to different parts of the network. Such allocation mechanisms are simple and transparent, but over time, are related only weakly to current need or use of the network.

Indirect assessment of needs; is used where are no reliable data for measuring need directly or where the cost of doing so would be disproportionate to the size of the budget being allocated. This method is used mostly for the allocation of budget to low cost/low volume roads, criteria used in the indirect assessment will include for example;

- The land area covered by the administration;
- Road density;
• Population;
• Industrial / agricultural production or potential.

These factors are weighted accordingly to their perceived importance, this approach provides a pragmatic basis for allocating funds in appropriate cases that is cost – effective, and acknowledges the socio-political aspects of the decision-making.

Direct assessment of needs: can be of different degrees of sophistication, as its most comprehensive, it will involve using the results of a detailed condition survey of all needs to determine work requirements. These are then costed to derive the budget requirement. This approach requires the use of treatment selection methods such as deterioration modelling.

Simpler methods: involve deriving norms for expenditure on roads in different hierarchies or of different surface types. Road lengths in each administration are simply multiplied by the relevant norms to give the budget allocation. Thus, there are several direct assessment methods. The methods chosen should suit the capacity of the level of government concerned. The key is to have clear objectives and then to appraise in a systematic way the extent to which each intervention contributes to realizing these objectives. It is often difficult to change existing allocation methods because there will be strong resistance from those who will lose out “political interest”. There may also be pressures to maintain a “regional balance” that may actually distort the optimal allocation of funds.

10 PRACTICAL BUDGETING TECHNIQUES

The purpose of resource allocation is to determine the appropriate total level of capital and maintenance investment that is to be made available for road repair and rehabilitation, bridge reconstruction and rehabilitation, and new construction, usually on an annual basis. Distribution is the manner in which total funds allocated for highway and bridge repair are made available to sub-national jurisdictions, road systems, and types of improvement.

Six identifiable patterns of resource allocation can be identified. The defining characteristic of these six patterns is the degree of shared responsibility between the Ministries of Finance and Transport (or their equivalents) in the allocation process.

In the first pattern, the responsibility for allocation, especially for the national road system, is retained in governmental hands. For example, the Ministry of Transport in Canada is totally responsible for resource allocation on the national road network. In Great Britain, the Department of Transport has the responsibility on a central as well as on a regional level. The procedure for resource allocation in the United States is very much reflected by the interaction of responsibilities between the Federal and State levels. In New Zealand, the government funding agency (NZTA) taken a leading role in allocating and distributing funds.

In the second pattern governmental jurisdictions are still in charge of allocation, while the distribution procedures are transferred to national, regional, and local road administrations. Germany, Japan, Norway, Portugal, Spain, and Switzerland belong to this allocation/distribution category.

In the third pattern, autonomous bodies are involved. This describes the allocation/distribution process in Italy, with the Autonomous State Roads Administration (ANAS).

Finland and Sweden represent the fourth discrete pattern. Although the financial responsibility remains in the hands of the government, the Road Administrations have a strong impact. This is consistent with the "management by objectives" philosophy that these countries have adopted.

The fifth pattern related to wealthy governments (economy based on oil & gas revenue) such as the case in Abu Dhabi, the financial approval issued by the Executive Council, the political decision basically controls the funding allocation and distribution.

The sixth pattern, developing countries, the overall planning process appears to be inconsistent and very far from optimal in meeting some sensible framework of objectives.

Typically, the Central Government generally defines the total annual roadway rehabilitation and maintenance budget. In addition to the initial budget allocation, the central government may also determine the distribution of those funds by governmental jurisdiction, road system and, in some cases, by major category of road improvement type, although the involvement of the central road authority varies by country.

Decisions are made using a combination of technical analysis to achieve efficiency in fund allocation, and political, social, technical and economic considerations to achieve funding equity and balance among competing interests and political jurisdictions. This combination of technical and political considerations appears to exist in some fashion. As was true with the degree of central government involvement in the allocation process, however, the variations among countries in the relative mix of technical and political considerations are broad.

Under the political level(s) the managing responsibilities of road administrations differ between countries. Some countries use "management by objectives", "directed autonomy", or "zero based budgeting" philosophies in carrying out their responsibilities; other countries are more directly tied to the Ministry of Transport which permits only "limited autonomy" to their road administrations.
11 PROBLEM STATEMENT

Road Controlling Agencies (RCAs) around the world are facing continuous challenges sustaining network funds. Given the fluctuation of market forces and instability of economies around the world. Effective management of a road network requires that budget levels are set at least sufficient to keep the core of road assets in a stable condition for long term. This requires that ongoing maintenance is funded, and that adequate provision is made for any strengthening works required. More than this minimum level will be required if the network is to be expanded or improved. Funding for road networks traditionally comes from governments (which is the case in Abu Dhabi – United Arab Emirates). It becomes increasingly more difficult to maintain road funding at past levels, one of the reasons is that typically roads are seen as a common good and funded as a social service.

Roads are usually funded through budget allocations determined as part of annual government budgeting processes. These allocations often bear little relationship to the levels of funding that road users actually contribute to revenue or to underlying needs of the network, measured in terms of economic criteria, consequently there is no direct linkage between revenue and expenditures. Two other facts relevant to Abu Dhabi are that the network is growing rapidly and will continue over at least the next 10 years, moreover the quality of final deliverables and new vested assets require immediate maintenance and additional operational expenditure, therefore there is a need to establish a robust long term assessment tool that caters for maintenance and operational requirements. This assessment should be based on life cycle costs throughout the useful life.

12 RESEARCH CONCEPT

The initial overall determination of budget allocation performed by the Ministry of Finance may be considered systemic in nature. It requires an objective and subjective evaluation of alternate investment strategies against a prescribed set of national or regional goals. In some cases, the initial distribution of funds by highway system and jurisdiction to achieve equity also involves measurement against these same or similar objectives.

Systematic measurement and evaluation requires the development of standardized data and analytical procedures to ensure that comparisons are accurately made throughout the nation's regions or provinces. The types of data required for this initial allocation are general in nature. They consist of measures of system usage and extent, land area, population, and other objective measures of a real dimension, as well as standard network measures that can be applied nationwide.

These requirements are defined within a strategic planning matrix arrived at through professional judgment, active consultation with districts within the country, or a combination of the two methods.

The essential difference between these comprehensive resource allocation strategies and the strategies typically used today is that standard resource allocation is a marginal process. Past year allocations are used as a baseline for comparison against possible budget options and evaluations are made on the basis of marginal changes in allocation and distribution. Under the more comprehensive method, budgets are "built up" on the basis of how well an allocation level or means of distribution achieves a prescribed goal or objective. The baseline seems to be a critical issue to Abu Dhabi, as the network is growing rapidly with increasing demand.

The distribution by type of road improvement is a rigorous engineering and/or economic analysis, requiring the use of sophisticated computer programmes that relate investment to system performance impacts. In most cases, distribution analysis is sufficiently sophisticated to relate investment to changes in measurable engineering parameters such as pavement and bridge condition, safety, or levels of service.

The quality, consistency, and application of complex road and bridge databases and analytical systems (asset management) to support the development of rehabilitation and maintenance budgets vary widely among participating countries. This information may include measures of roughness, deflection, rideability, and/or surface cracking. This information can be used to establish bearing capacities to support the development of pavement management programmes.

On the other hand, drainage adequacy and subbase condition is seldom available, and these are major factors that help determine the particular type of pavement rehabilitation strategy required for accurate life-cycle pavement cost estimation. In addition, future travel forecasts, particularly by vehicle category and subcategory.

This move toward consistency requires the development of standard techniques and data systems, within the context of a fully integrated road and bridge management system. The system should be capable of accommodating the types of allocation and fund distribution currently required, including:

- Development of budget totals based on relating expenditures to changes in overall system performance;
- Development of regional distributions through the use of economic analysis that equitably
- Compares the overall value of the investment by jurisdiction; and
- Development of functional distribution tools to calculate and compare changes in road user costs associated with various investment strategies.

13 THE WAY FORWARD
Further work is in progress, including the theoretical framework, system design and proposed system architecture which will improve the credibility of the budgeting process. The system platform will be based in various data sources such as valuation, predictive modelling, long term & annual plans together with other source of data. More details will be presented during the conference as the study is progressing and further outcomes are achieved. Further update on this paper regarding the setting matrix, data gathering, analysis and initial output can be submitted within the next few weeks, the following screen shots illustrate the front end of the system currently under development and based on the research findings.

14 ACKNOWLEDGEMENTS

The author would like to acknowledge the support and guidance received from the research supervisors, Prof. Mark Porter and Prof. John Yeaman.

15 REFERENCES


16 BIBLIOGRAPHY

Biographical note for Alan Roland

Alan is a Chartered Professional Registered Engineer of New Zealand (MIPENZ), (CPEng), (IntPE) and REAAA. Member of INGENIUM (Association of Local Government Engineers of New Zealand). Alan holds a bachelor degree in Civil Engineering, and Masters Degree in Transportation Asset Management. He has experience almost 30 years in transportation asset management at both strategic and operational levels; infrastructure development, design, operation and maintenance; development and implementation of road asset management processes and systems. Alan presented a number of asset management papers at the national and international workshops and conferences.

Alan has been working with government agencies, consultants and contractors, and currently is working with the Department of Transport in Abu Dhabi, as the Network Inventory and RAMS Specialist. Alan is currently a PhD candidate and currently working on this research.

Biographical note for Mark Porter

Mark Porter is a Professor of Engineering at the University of the Sunshine Coast, where he is responsible for the development of new teaching and research programs in Civil and Mechanical engineering. He is a water resource and environmental engineer with a strong interest in engineering education and a background in research, teaching and academic management. He is now overseeing the development of new research programs in asphalt pavement engineering, permeable pavements, and the impacts of climate change on coastal infrastructure.

Mark's achievements include a university Excellence Award for Research (1993) and an Excellence Award for the Design and Delivery of Teaching Materials (2003) and a national Carrick Award for outstanding contributions to the Enhancement of Learning (2007 - Learning and Teaching Category: joint winner). He is a Fellow of the Institution of Engineers and held the position of Chairman for the Sunshine Coast Local Group of EA in 2012-13.

Biographical note for Professor John Yeaman

Professor John Yeaman FTSE, PhD, FIE (Aust) CPEng, RPEQ is the Director of The Queensland Functional Pavement Centre at the University of the Sunshine Coast and Professor of Civil Engineering Construction.

John has been in the Civil Engineering Materials and Paving industry since 1957 directly involved in new and innovative products and services for 25 of those years. He built his own business in Pavement Management in which he was the principal engineer for 30 years.

For the past 3 years his has been an academic heavily involved in Pavement design, construction, operation and renewal maintenance, contract management and project management.
Title: FUNCTIONAL LIMITS OF THE W-BEAM GUARDRAIL

Authors: Göran Fredriksson,
Swedish Safety Barrier Association
Box 3010
720 03 Västerås
Sweden

Hans G Holmén
Swedish Transport Administration
Röda vägen 1
781 89 Borlänge
Sweden

Abstract
The W-beam (A-profile) guardrail is common all over the world. A tested, approved and properly installed product causes little concern, or does it? What about SUV’s or sports cars not included in the standards? Even the “standard” car could be in trouble as the original position of the guardrail can change over time. Where are the functional limits of an ageing guardrail, where it no longer can provide the road safety features you should expect? To find out, a research project was financed by the Swedish Transport Administration. It was possible to establish several of these limits using full scale tests and it also provided an opportunity to compare initial computer simulation results of unusual test collisions with the real outcome. The results revealed functional limits when positioning the guardrail too low and/or leaning outwards. It showed how a small change in position can make a big difference in the outcome of a collision for “non-standard cars”. Poor ground conditions was tested as well as two serious types of damages. For comparative reasons, the TB32 test according to SS-EN 1317-2 is performed for all tests, but excluding measurement of working width and ASI. The “standard” guardrail with W-beam and weak sigma-shaped steel posts at 4 m spacing was used.

Introduction
The ageing of road equipment in general and particularly the functional degrading of safety barriers/guardrails is a major concern for the road owner. In an ideal situation the ground conditions, installation and guardrail position would all be and remain “as tested”. Later, when a vehicle impacts the guardrail, that vehicle is preferably similar to those stated in the standard requirements found in SS-EN 1317-2 [1] and used during the Initial Type Test (ITT). We all know that in reality these factors will all vary and it is a known fact that over time guardrails tend to be positioned lower than their nominal height over the road surface. This height deviation is sometimes combined with the posts leaning outwards from the roadside. The causes of this are typically:

- Gradual lowering of the guardrail as the posts gradually sink deeper into the soil.
- New paving being performed on the road with no corresponding height adjustment of the guardrail.
- A combination of the above.
- Tilting outwards can be caused by frequent pressure from snow removal operations.
- Light, narrow angle collisions.
- Compression of the soil under the road causes deformation of the substructure pressing it out towards the sides.

During installation of a guardrail, there will be other factors that deviate from those present at the test site when the ITT was performed. Many of these deviations are unfortunately not visible when the work is finished, for example soft/loose soil conditions or shallow bedrock, where the installers might have cut off the posts instead of using recommended methods such as drilling a hole in the rock. Maintenance contracts calls for corrective action, but are the current given limits and instructions (if any) where this action has to be activated really correct? What happens to the guardrails functionality when deviations such as wrong positioning, loose soil conditions or “non-standard” vehicles are present? Apart from the natural gradual ageing of the guardrail, damages to the W-beam and/or to the post will happen. Mechanical damages such as dents or scratches are a common sight and occasionally there will be a more serious damage like a vertical tear in the W-beam or a
post missing/cut off. How does this affect the functionality and what priority should be given to maintenance crews for repair? These are central questions for any road owner and the need for guidelines is evident. Therefore The Swedish Transport Administration (STA) decided to launch a project in order to investigate how the functional limits of the W-beam guardrail is affected when various deviations from the nominal tested barrier are introduced as well as compatibility with non-(EN1317)standard [1][2] vehicles. From the knowledge acquired it would be possible to write more specific maintenance contracts and give instructions that are based on results from real full scale tests. A spin-off effect would be the parallel and unique opportunity to compare results from less costly computer simulations of non-standard impacts to the outcome in physical tests.

Background
Vehicle Restraint Systems, VRS, are required to be CE marked and as from 1st of July 2013 this is also a legal requirement within the European market. CE-marking of construction products is obtained by the fulfillment of certain requirements. These requirements are set down in a harmonized standard related to the specific product and its intended use. The requirements for vehicle restraint systems are specified in the harmonized standard EN 1317-2, which consists of 15 steps, each and every with raised demands on containment of vehicles (containment levels), from the smallest 900 kg passenger car to, at the other end of the scale, a 38.000 kg semitrailer HGV.

In Sweden, STA has chosen the containment level N2 as a basic requirement for a majority of the roads. The level N2 is tested with a 1500 kg passenger car vehicle travelling at 110 km/h into the guardrail at an angle of 20 degrees. (The specific test is given the name TB32 inside the document EN1317-2, which is why it is often referred to as a TB32 test.) The guardrail shall contain and redirect the vehicle, with certain restrictions on the accelerations measured inside the vehicle, the deflection and strength of the guardrail as well as on the vehicle behavior during and after contact with it. Given that all recently installed VRS (at least most) are tested according to this N2-level of EN1317-2, it was convenient for this project to use that particular test as a kind of starting point or calibration point, making reasonable variations from that tested and approved configuration. For reasons currently unknown, it was once decided to use 550 mm above ground to the centerline of W-beam as a standard height for guardrails in Sweden, as opposed to approx. 605 mm in many other European countries.

This height is still guiding even when new guardrail designs are brought to the market. The lower initial position of the guardrail reduces the margin to the lower functional limit and could possibly also have other implications on the expectations of safe behavior when impacted with other type of vehicles, such as Sport Utility Vans (SUVs) or Multi Purpose Vehicles (MPVs). A paper, “Midwest guardrail system for standard and special applications”, submitted to Transportation Research Board, TRB, 2004 [3] describes the historic development of the Midwest guardrail system and how increased guardrail height, amongst other measures, improves the performance of the guardrail. The original height of 530 mm to W-beam profile centrelines was adjusted to 550 mm during metrification (706 mm to top) and then to 631 mm (787 mm top mounting height) after research that was done in the year 2000 by Midwest Roadside Safety Facility [4][5][6]. It shows that vehicles with a high centre-of-mass as well as smaller ones can be contained and safely re-directed with the increased barrier height.

Even though the Midwest guardrail system is a strong post system using blockouts (mainly to reduce the risk of wheel snag on the posts) and sometimes wooden instead of steel posts, the results clearly indicates that a similar improvement can be achieved with the weak post system without blockouts.

Methodology
To be able to compare the results from collisions with a deviating guardrail and/or vehicles from to those initially performed as part of the ITT, it was decided to replicate the original test set-up for the project collisions. The guardrail chosen was the common and well-known W-beam (A-profile) as it was originally tested by STA (at that time called Swedish Road Administration) in 1995 at the Swedish National Road and Transport Research Institute, (VTI). The crash test that is simulated and subsequently performed physically is SS-EN 1317:1998 test TB32, i.e. a 1500 kg car impacting the guardrail at a 20-degree angle. The impact speed is 110 km/h. The guardrail model that is studied is the formerly Swedish Road Administration standard barrier known as EU4 [7]. W-beam thickness is 3 mm with 4 m post spacing (S-shaped posts). The computer simulation was performed with the FE program LS-DYNA [8]. The pre-processing was done with ANSA [9] and LS-Pre-Post [10]. An LS-Pre-Post was used for the post-processing. All physical tests and simulations use the same barrier set-up i.e. the barrier is 76 m long at full height (nominal 550 mm), see fig. 1 below, and 2 x 12 m sloped anchored terminals. The point of impact is 20 m from the nearest end of the barrier.
as in the original tests. For validation of the simulation model, a modified Ford Taurus 1991 was used as a vehicle model [11].

Figure 1. Test set-up as described in EN 1317. Exit box and pass or fail as shown was modified for the project.

It was decided to exclude measurement of impact severity, as it is not a part of the pass or fail criteria as stated above. Such a decision would help in keeping costs down as well as reducing preparation time in between collisions. Pass or fail would be based only on over-ride of the guardrail, vehicle roll-over, or any other hazardous behavior. To decide on initial positions to be tested, computer simulation was used when suitable to find out where the limits seemed to be. Depending on the outcome it was then decided on how to proceed. Results from original tests performed in 1995 was used to validate the simulation model of the EU4 barrier used in the project to find the critical pass or fail positions.

For the first tests of deviating positions (low, inclined, low and inclined) a total of 40 simulations were performed (more than anticipated). Simulation of collision results is necessary to be able to determine in theory where the functional limits of the barrier seem to be and from that decide on the physical test. From the point of keeping within the project budget, minimizing the number of costly full scale impacts is vital and reliable simulation results were crucial to be able to make the right decisions, especially regarding initial barrier positions to be tested in the different cases. The Guardrail position and parameters studied are defined as shown in figure 2 below.

Figure 2. Description of parameters in the parameter study. Protrusion is the length of the post sticking out above ground when the barrier is in an inclined position. Nominal protrusion is 550 mm.

The term "effective height" was established by the project team to describe the perpendicular distance between the centre-line of the horizontal beam and the ground level when the guardrail was mounted with an inclination and with variations in protrusion. In figure 2, barrier position C above, this is illustrated. The term is useful as a reference to compare the height of the centre-line when the guardrail is in a perpendicular position to the height achieved with the guardrail at an inclined position and post protrusion varies. During simulation the pass criteria was that the guardrail contained and re-directed the vehicle and the fail criteria that it over-rod. The case of the vehicle rolling over after being contained was not included although this tendency could be seen in some cases.

The results from the simulations showed that a guardrail mounted too low, but perpendicular, ceased to contain the vehicle at a height of 450 to 400 mm which is a 100 to 150 mm lower than nominal.
Further simulations of an inclined guardrail indicated that when the barrier approached a 40° angle, the limit for pass criteria was exceeded. However, when inclination and low position was combined in the simulations, it was not that clear where the critical position was. A deviating point appeared among the “pass-points” and it was uncertain why this occurred, but it indicates that the guardrail behavior is not robust in the sense that it is at, or close to, the limit where it cannot anymore reliably uphold its function.

Simulation points and their outcome as pass (green) or fail (red) are presented in figure 3:

![Figure 3. Parameter study results](image)

The next task for the project team was to create a Design-Of-Experiments (DOE) for the subsequent physical tests. The DOE was created based on the experience gained from the simulations, i.e. the parameter study. The strategy was to determine the x and y axes of the “Decrease in post protrusion” vs. “Inclination” diagram with a minimum amount of physical tests. Optimally, only four tests would be needed to determine the x and y axes with reasonable accuracy leaving the remaining tests to the, what it seemed, more complicated case with combined “Decreased protrusion” and “Inclination”. Experience from the simulations showed that the x and y axes would be rather straight forward to determine, i.e. we needed one green “pass” point and one red “fail” point on each axis of the diagram. The simulation results for the combined cases were more difficult to interpret. There was one deviating “red” point among the green points, see figure 3 above, indicating sensitivity in the guardrails behaviour.

The final DOE for physical tests of the guardrail in deviating positions is shown in figure 4.

![Figure 4. Tested guardrail positions, numbered in the order they were performed.](image)

Based on the knowledge gained so far the project moved on to test impacts with vehicles that the guardrail is not specifically designed for, especially SUV’s. There is a growing concern that the common N2 barrier will not retain these vehicles properly and might even cause rollovers. The increasing number of this type of larger, heavier and higher vehicle on the roads has been noticed by road authorities, and there is a need for more knowledge in the matter. In the TRL Report TRL658 [12] it is noted that the market-share in the UK of this type of vehicles has doubled over the last 15 years (until 2007). The analysis performed on real world crashes indicated a relatively high incidence of rollovers when hitting a barrier and that vehicle height, impact and speed correlated well with the rollover risk. Given the lower nominal height (550 mm to centreline of W-beam) of the W-beam “standard” guardrail in Sweden it was decided to perform the TB 32-test this time using a SUV. The choice was a Volvo XC90, which is one of the most common models of SUV’s in Sweden. Simulation of the
impact was more of a challenge here, as there is no suitable computer model of the XC90 publicly available. Instead the simulation team had to use a modified computer model from Ford explorer. Depending on the outcome (fail at height 550 mm was assumed) the next step would be to raise the barrier approx. 100 mm to 650 mm. With this increase “pass” is expected for the XC90 and if so, what will happen if a small sports car hits a barrier of that height? The choice of sports car fell on one of the most common models in the world, Mazda MX-5 “Miata” and again the simulation team noted that no computer model was available, see figure 5 with photos of the tested car models.

![Volvo XC90 and Mazda MX-5 as used in the full scale tests.](image)

The car models used in the simulations were obtained from the website of National Crash Analysis Center (NCAC) at The George Washington University in Virginia. These car models had to be modified, i.e. weight and center of gravity (COG), to fit the cars used in the physical experiments as close as possible. The Volvo XC 90 used in the physical test was substituted for a Ford Explorer and the Mazda MX-5 for a Suzuki Swift in the simulations. The cars are similar in terms of dimensions, but are of course not identical. The structural composition of the cars used may also differ but this has not been addressed [13]. The next step was to investigate the functional limits when the guardrail installation is not properly done or there are damages to the W-beam or post. From experience two unfortunately well-known deviations during installation was chosen for full scale tests; loose soil (compared to normal test-site conditions) and a shortened post. The TB32 test and the EU4 guardrail is used as done so previously, but this time without prior simulation. For the final tests, two typical and serious damages were introduced, a post cut off at ground level (can happen during snow removal for instance) and a vertical tear in the W-beam. In both cases the outcome can serve as a basis for repair priorities given to maintenance crews. The vertical tear was simulated to find out where the limit could be before the W-beam is fully torn apart during impact. The material damage and rupture was modelled using material model 81-82 in LS-DYNA, denoted *MAT_PLASTICITY_WITH_DAMAGE. The parameters were optimised and calibrated against multiple crack specimens cut out from a W-beam, in which cracks of different lengths were introduced. The effective plastic strain at which material softening begins and the effective plastic strain at which material ruptures were the two parameters that were optimised by inverse modelling using LS-OPT. The damage curve was assumed quadratic in-between these two values and the mesh density used in the modelling of the test specimens was the same as in the final W-beam application. The optimal result showed an excellent agreement to the experimental force-displacement test specimen curves, see figure 6 below.

![Force-displacement curves from the crack test specimens (indicated by markers) and corresponding curves from simulations using the optimal material failure parameters found in the calibration (green curves).](image)
In addition to the description above, it should be noted that simulations using the calibrated model was used in order to determine an appropriate value of the initial crack length in the performed physical test. The agreement between the simulations and the performed physical test was acceptable, although the simulation was shown to be slightly conservative.

**Results**

Note: The nominal tested position of the W-beam guardrail in Sweden is vertical with a distance of 550 mm between ground level and centerline of W-beam (equals approx. 700 mm to top of post/W-beam).

Unless otherwise stated, the vehicles used were standard Volvo 940 or 850 which are considered equal from a physical test point of view. The vehicles were carefully calibrated by weight to pass the criteria of EN1317-1:2010 for the TB32 vehicle, i.e. 1500 kg ± kg and a given COG.

**Low guardrail**

# 1) TB 32 test. Vertical position. Post protrusion reduced 150 mm to 400 mm
**Result:** FAIL The car passes over the guardrail and rolls over on the other side

# 2) TB 32 test. Vertical position. Post protrusion reduced 100 mm to 450 mm
**Result:** PASS The car is retained although with a heavy roll

**Inclined guardrail**

# 3) TB32 test. Inclined position approx. 41.5°, effective height 450 mm, see figure 7.

![Effective height, Eh](image)

Figure 7. Illustration of the term “effective height”, Eh, above ground.
**Result:** FAIL The car passes over the guardrail and rolls over on the other side.

# 4) TB32 test. Inclined position approx. 30.8°, Eh 500 mm, see figure 7.
**Result:** PASS The car is retained in a controlled manner.

**Low and inclined guardrail**

# 5) TB32 test. Inclination approx. 30.8°, combined with a reduced post protrusion of – 60 mm, Eh 445 mm.
# 6) TB32 test. Inclination approx. 25°, combined with a reduced post protrusion of – 50 mm, Eh 473 mm.
# 7) TB32 test. Inclination approx. 18°, in combination with a reduced post protrusion of – 50 mm, Eh 490 mm.
**Result:** FAIL in all the cases above. The car passes over the guardrail and rolls over on the other side

# 8) TB32 test. Inclination approx. 9°, in combination with a reduced post protrusion of – 50 mm. Eh is now approx. 503 mm. Figure 8 below illustrates the actual position of the guardrail as tested.

![Tested position in darker grey, correct nominal position in light grey. Photo taken prior to test.](image)

**Result:** PASS/FAIL. Guardrail retains the car (PASS), but it is thrown upwards on the side after impact, lands and rollover 3 times (FAIL)
Non-standard vehicle impact: SUV, standard guardrail 550 mm
# 9) TB 32 run using Volvo XC90 incl. dummies 2 adults and 3 children, weight 2322 kg, speed 104.6 km/h.
Guardrail correctly mounted.
Result: FAIL. The car passes over the guardrail with a noticeable roll.

Non-standard vehicle impact: SUV, higher guardrail 650 mm
# 10) TB 32 run using Volvo XC90 incl. dummies 2 adults and 3 children, weight 2318 kg, speed 104.4 km/h.
Guardrail correctly mounted but height increased with 100 mm.
Result: PASS. The car is retained, comes out with a roll but stays on its wheels in an orderly behavior.

Non-standard vehicle impact: sports car convertible, higher guardrail 650 mm
# 11) TB 32 run using Mazda MX-5, weight 1124 kg, speed 104.7 km/h. Guardrail correctly mounted but height increased with 100 mm.
Result: PASS. The car is retained and comes out of the impact along the guardrail in a controlled manner.

Shortened posts, length approx. 1.250 mm
# 12) TB 32 test. Vertical position, standard height 550 mm but 3 posts originally 1950 mm cut of 700 mm to 1250 mm total length. Impact point between 1st and 2nd post.
Result: PASS. The car is retained although one post is pulled out of the ground and thrown. The car also comes out with a considerable yaw to end up with a 360° rotation. Contact with guardrail during impact is longer than with standard posts.

Loose soil, natural gravel (cannot be compacted)
# 13) TB 32 test. Vertical position, standard height 550 mm and posts, L= 1950 mm total length. Anchoring still remains fixed in standard soil (as present at test site).
Result: PASS. The car is retained with a controlled behavior, although 3 posts are pulled out of the ground.

Post cut off at ground level
# 14) TB 32 test. Vertical position, standard height 550 mm but one post cut off completely at ground level.
Impact point approx. 3 m before the cut of post.
Result: PASS. The car is retained with a controlled behavior. The cut off post is thrown away by the impact.

W-beam with 155 mm vertical tear
# 15) TB 32 test. Vertical position, standard height 550 mm. A vertical tear with a curved length of 155 mm in the W-beam is introduced (approx. 1/3 of the cross section of the W-beam). Simulation had shown that the W-beam should not be torn apart with a tear like this. “Worst” impact point for load on the tear according to the simulations is 5650 mm in front of the tear which the test collision set-up also tried to duplicate.
Result: PASS. The car is retained with a controlled behavior and the W-beam is not torn apart. The tear increases approx. 50 mm

Figure 9. The vertical tear of 155 mm just before the test vehicle reaches it. Impact point approx. 5.5 m upstream.

The physical test described above are presented in more detail in the written reports from VTI [14] [15].
Conclusions

It is important for the following to bear in mind that some parts of the guardrail used in the tests are still the original 76 m from the first test run and that the sloped 12 m anchored terminals remained the same during all the consecutive test runs. After each test the guardrail was inspected and all damaged parts were exchanged. This meant that the parts of the guardrail that remained was gradually stretched out, especially at the anchor points, due to the load of the impacts. Compared to a common guardrail found along any road the tested can most certainly be said to be more rigid in the sense that it has been pre-stretched, especially for the later tests. Figure 10 below show how this had to be compensated at one of the joints after a number of test collisions.

Figure 10. New holes made in one of the overlap joints to compensate for the gradual stretching of the guardrail.

In general the W-beam guardrail is quite forgiving to deviations of various kinds, as can be expected of a product that has been present in real traffic for more than 50 years and spread around the globe. However, there are definitely cases where the safe functionality is impaired and a false sense of security is given to road users. From the tests performed and the results obtained, the following can be said about the W-beam guardrail in terms of functionality to maintain and achieve containment level N2:

- A vertical guardrail ceases to function at a height below 450 mm from ground to W-beam centerline.
- A leaning guardrail must not exceed an inclination of 15° from roadside with full protrusion of post (= nominal as given by the manufacturer) measured from ground level. (15° is recommended due to the difficulty in determining full post protrusion, which also can be combined with other factors affecting the outcome of a collision in a negative way).
- The combined deviation of decreased height and guardrail inclination is devastating to the guardrail functionality. Decreased post protrusion exceeding 50 mm combined with a maximum inclination of 5° outwards from roadside needs to be adjusted without further delay.
- The W-beam guardrail can handle impacts in a reasonable way although one post is cut off/missing. The same thing can be said if some posts, at the place of impact, are mounted in loose soil. Other road users will be at risk though, as there is high probability of posts being pulled out of the ground and thrown around.
- Vehicles of SUV-type with considerably higher weight (2000 kg +) than the N2-test 1500 kg car, higher COG and ground clearance, will most likely not be retained by the conventional 550 mm high guardrail. This is based on a N2-type collision, but simulations also showed that at a more real-life impact angle of 10°, the vehicle is retained by the guardrail. This can be one of the explanations to the fact that rollover accidents with SUV’s impacting guardrails are not more frequently recorded.
- A vertical tear is a serious defect and a damaged W-beam should be replaced with high priority, but it can also be reassuring to know that even though 1/3 of the cross section can be torn, that it is likely to retain a car at N2 level without coming apart.

Improvements to guardrail functionality:

- It can be good practice to have installation tolerances that are only positive regarding the guardrail height, i.e. allows higher than nominal heights.
- A guardrail height of 650 mm improves functionality and traffic-safety for larger groups of road users as it proves capable of handling both bigger and heavier vehicles (SUV-type) as well as smaller sports cars in a safe manner, apart, of course, from the standard N2 vehicles.
- Attention to good anchoring of terminals and well tightened overlap joints are key factors in achieving safe and predictable function during impact.
Simulation, as performed in the project, proved to be very accurate in predicting the outcome of the real test collisions and a valuable tool for DOE also when working with cases outside the standard EN-1317 scope. Validation of simulation models against results from real tests and knowledge of actual material properties in the guardrail are two other important ingredients to reach reliable simulation results. With all this in place, simulation can be used and trusted when investigating unusual collisions, vehicles or developing new guardrail designs. In real life situations, the geometrical situation (for example old winding roads, bridges under preservation, etc.) or other considerations (such as protection of trees/plants/buildings) sometimes prevents conventional proven guardrail installation. In such cases simulation is probably the most beneficial way to find an economic and optimal solution looking at traffic safety for all road users. There are also limitations to computer simulation, where two were noted during the project, lack of suitable computer models of vehicles in some cases and, if the soil is loose (poor soil conditions) compared to normal test site conditions, then additional soil tests are needed in order to include this effect in the model.

References

**PAPER TITLE**  
Guideline for Road Safety Countermeasures

| TRACK |  
|---|---|---|---|
| AUTHOR (Capitalize Family Name) | POSITION | ORGANIZATION | COUNTRY |
| Yuta OZAKI | Researcher | National Institute for Land and Infrastructure Management | JAPAN |

<table>
<thead>
<tr>
<th>CO-AUTHOR(S) (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
</table>
| Nozomu MORI  
Susumu TAKAMIYA  
Sho KAMIYA | Director  
Head  
Guest Research Engineer | National Institute for Land and Infrastructure Management | JAPAN |

| E-MAIL (for correspondence) |  
|---|---|
| ozaki-y82ac@nilim.go.jp |

**KEYWORDS:**  
Road safety countermeasure, Black spots, Database

**ABSTRACT:**  
Traffic accident casualties in Japan have been decreasing in number, with fatalities falling to 4,373 in 2013. But there are still many deaths due to traffic accidents. Therefore, more efforts to reduce accidents are necessary. Because traffic accidents tend to occur frequently at certain spots, it is thought that road traffic conditions are an indirect cause of accidents. Therefore, countermeasures by road administrators are important. In Japan, in order to make traffic safety countermeasures more effective, road administrators identify accident black spots and analyze accident factors in order to take effective countermeasures. Because accident factors vary between accident types and road traffic conditions, a great deal of knowledge and experience are necessary to accurately analyze accident factors. So in order to share road administrators’ experience with road traffic accident countermeasures, we have collected and organized accident factors and patterns of countermeasures taken in past traffic accidents into the Guideline for Road safety Countermeasures. This Guideline lays out accident occurrence processes and factors related to each accident type. It also presents road traffic conditions in which each accident type is likely to occur, as well as examples of effective countermeasures against them. This paper introduces this Guideline.
Guideline for Road Safety Countermeasures

Yuta OZAKI, Nozomu MORI, Dr. Susumu TAKAMIYA, Sho KAMIYA

1 National Institute for Land and Infrastructure Management, Tsukuba, Ibaraki, JAPAN
Email for correspondence: ozaki-y82ac@nilim.go.jp

1 INTRODUCTION

Traffic accident casualties in Japan have been decreasing in number, with fatalities falling to 4,373 in 2013. But there are still many deaths due to traffic accidents. Therefore, more efforts to reduce accidents are necessary. Because traffic accidents tend to occur frequently at certain spots, it is thought that road traffic conditions are an indirect cause of accidents. Therefore, countermeasures by road administrators are important.

In Japan, in order to make traffic safety countermeasures more effective, road administrators identify accident black spots and analyze accident factors in order to take effective countermeasures. Because accident factors vary between accident types and road traffic conditions, a great deal of knowledge and experience are necessary to accurately analyze accident factors. So in order to share road administrators’ experience with road traffic accident countermeasures, we have collected and organized accident factors and patterns of countermeasures taken in past traffic accidents into the Guideline for Road safety Countermeasures. This Guideline lays out accident occurrence processes and factors related to each accident type. It also presents road traffic conditions in which each accident type is likely to occur, as well as examples of effective countermeasures against them. This paper introduces this Guideline.

2. STATUS OF TRAFFIC ACCIDENTS AND ROAD SAFETY COUNTERMEASURES FOR ARTERIAL ROADS IN JAPAN
2-1 Status of Traffic Accidents in Japan

After the end of World War II, Japan entered into an era of rapid economic growth, which accelerated motorization and caused a remarkable rise in car ownership. Car ownership soared in the decade from 1960 to 1970, from about 3.4 million to about 19 million. Consequently, fatalities by traffic accidents sharply increased. In 1970, the annual fatalities by traffic accidents was 16,765, which was a record high car accident fatalities in the history of motorization in Japan.

In 1970, the Traffic Safety Basic Act was formulated, and the Traffic Safety Basic Plan was prepared based on the Act as Japan’s first comprehensive traffic safety countermeasures. Under the Traffic

![Graph showing changes in traffic accident casualties, fatalities, and vehicles from 1945 to 2015.](image)

Figure 1. Changes in the death toll, casualties, casualty accidents, and cars owned in the entire country
* Data from National police Agency (NPA) and Ministry of Land, Infrastructure, Transport and Tourism (MLIT)
Safety Basic Plan, a variety of road-based traffic safety countermeasures were taken, including the construction of expressways, sidewalks, and pedestrian bridges, as well as the installation of guard fence, having made great contributions to reduction in car accidents.

The number of car accidents increased temporarily as a result of the subsequent rise in traffic volume. In recent years, however, the fatalities and casualties by traffic accidents and the number of fatal and casualty accidents are all decreasing. The annual death toll in 2013 is 4,373, which is about one fourth that in 1970 (see Figure 1).

Figure 2 shows the ratios of the road lengths, the number of fatal accidents and the number of casualty accidents by the type of road. National highways and prefectural roads, excluding expressways, are referred to as arterial roads, while residential roads, which are defined as city, town, or village-managed roads used primarily for people’s daily living. While arterial roads occupy a small portion of total road lengths, they occupy a high proportion in the number of casualty accidents. Arterial roads occupy a much higher proportion in the number of fatal accidents than the casualty accidents. This data indicates that car accidents on arterial roads tend to result in fatalities (see Figure 2).

![Figure 2. Ratios of the road lengths, the number of fatal accidents, and the number of casualty accidents by the type of road](image)

* Data from NPA and MLIT

2-2 Safety Countermeasures against Traffic Accidents on Arterial Roads

In Figure 3, the road segments, namely the road segment with straight road and the road segment with intersection, of arterial roads, excluding expressways, are arranged in descending order of the casualty accident ratio to indicate the severity of the casualty accident ratio. The horizontal axis of the graph is the rank in the casualty accident ratio for each segment divided by the total number of segments. The vertical axis of the graph presents the casualty accident ratio (= Number of accident / kilometrage). For the road segments with straight road, all such road segments in the entire country are divided by a length of 200 m to 1,000 m, which amounts to a total of about 500,000 road segments with straight road. For intersections, a segment comprising a single intersection is counted as one road segment with intersection. The casualty accident ratios were calculated from accidents that occurred from 2009 to 2012.

In arterial roads excluding expressways, accidents occurred in about 50% of all the road segments for both road segments with straight road and road segments with intersection. The casualty accident ratio is particularly high at certain locations, which means those locations are highly accident-prone spots. It is thought that at those spots road traffic environmental such as road structure or traffic conditions are indirect causes of accidents. It is important for the road administrator to take appropriate countermeasures to prevent traffic accidents, such as road improvement, at those spots.

The road administrator, the Ministry of Land, Infrastructure, Transport and Tourism (MLIT), identifies those locations with particularly high casualty accident ratios as “black spots” and intensively conducts various countermeasures. Black spot countermeasures aims to reduce the accidents by about 30% of all the accidents occurring in those spots.
Figure 3. Casualty accident ratios at road segments with uninterrupted flow and with intersections

*Data from NPA and MLIT

Figure 4 shows the effects of black spot countermeasures. A total of 17,116 casualty accidents occurred at black spots before implementation of the countermeasures. After implementation, the number of accidents was reduced to 12,265, which is 28.3% lower than before implementation of the countermeasures. Considering the fact that the number of casualty accidents on roads in the entire country increased by 1.8% from before implementation of the countermeasures, the effectiveness of the countermeasures is interpreted as a 30.7% reduction in the number of casualty accidents from before implementation. This fact confirms that the accident reduction target by the countermeasures, about 30%, has been successfully fulfilled.

Figure 4. Effects of countermeasures taken at high risk spots

*Data from NPA and MLIT

Figure 5 shows the number of locations by degree of reduction arranging the accident reduction ratios by location. While there are many locations where accidents were reduced by over 50%, there are others where accidents were not significantly reduced or where accidents increased in number. It is necessary to steadily reduce accidents at locations where countermeasures are taken in order to enhance the effectiveness of the road safety countermeasures. It is, however, up to the experience and knowledge of each road administrator whether or not accurate countermeasures that match each location can be developed and implemented to realize accident reduction. Under these circumstances, it was decided that the Guideline for
Road Safety Countermeasures be established as a tool to realize the sharing of knowledge and experience of engineers and specialists.

3. GUIDELINE FOR ROAD SAFETY COUNTERMEASURES

3-1 Preparation of the Guideline for Road Safety Countermeasures

Promotion of road safety countermeasures is recommended according to the following procedures in Japan:
1) Determine (1) the “accident type” for accidents to be reduced by means of road safety countermeasures.
2) Estimate (2) the “process of accident occurrence” to clarify how an accident of the form determined above occurred.
3) Estimate (3) the “accident factors” involved in the process of accident occurrence estimated above.
4) Determine (4) the “action policy” to cope with the accident factors estimated above.
5) Develop (5) “countermeasures” based on the action policy determined above.

Accurate implementation of road safety countermeasures requires accurate estimation of the process of accident occurrence or accident factors as mentioned above. To this end, it is necessary to comprehensively assume the process of accident occurrence or accident factors based on the form of accident and select the likely ones among them that are considered reasonable relative to local conditions. It was decided that the Guideline for Road Safety Countermeasures be created by collecting and analyzing the past road safety countermeasures, comprehensively analyzing the process of accident occurrence or accident factors assumed for each form accident type, and organizing action policies and countermeasures considered effective to deal with each accident factor. To realize this, the information registered in the accident countermeasures database was used to prepare the Guideline. The accident countermeasures database is a database of information on the content of countermeasures that were taken, and the data of accidents that occurred at locations designated as high-risk spots.

The database also contains the information related to the basic principles of road safety countermeasures, namely (1) the “accident type” of accidents to be reduced by means of appropriate road safety countermeasures; (2) the “process of accident occurrence” to clarify how an accident of the form occurred; (3) the “accident factors” involved in the process of accident occurrence; (4) the “action policy” to cope with the accident factors; and (5) “countermeasures to be implemented” based on the action policy.

The Guideline was prepared according to the procedure consisting of i) review of the composition of the Guideline, ii) organization of the countermeasures taken in the past and registered in the accident control database according to the composition reviewed as in i), and iii) elimination of the countermeasures that failed to produce successful effects or action policies considered inappropriate.

The Guideline was composed so that the contents of the Guideline can help implementers of road safety countermeasures smoothly follow the procedures of the field review of the countermeasures at actual
locations. To be specific, the contents are organized in a tabular form in the same sequences as the field review procedure, or (1) accident type, (2) the process of accident occurrence, (3) the accident factors, (4) the action policy, and (5) the type of countermeasures. The contents are summarized sequentially from the accident types in a tree format. Figure 6 shows a sample composition of the Guideline.

In analyzing the factors of traffic accidents, while it is difficult to identify factors related to road users such as drivers after the accident occurred, indirect causes related to road structure that may induce mistakes on the part of road users can be identified by checking the road structure even after the accident. Considering this fact, accident factors are described in two ways. Take (3) accident factors for instance. They are divided into (3-1), or the mistakes of road users including drivers, such as “insufficient safety check” or “speeding,” and (3-2), or remote factors related to the road structure as contributory to those behaviors, such as “difficult to make safety check due to obstacles” or “downslope.” This arrangement therefore allows the users of the Guideline to estimate “accident factors” by checking the type of road structure that greatly helps road users make safety checks.

The last step of the procedure is iii) elimination of the countermeasures that failed to produce successful effects or action policies considered inappropriate.

<table>
<thead>
<tr>
<th>Process to be studied</th>
<th>Classification of accident occurrence</th>
<th>Event of accident occurrence</th>
<th>Analysis of accident factors</th>
<th>5. Consideration of action policy</th>
<th>6. Production of the measures to be implemented</th>
</tr>
</thead>
<tbody>
<tr>
<td>Accident type</td>
<td>Classification of accident occurrence</td>
<td>Event of accident occurrence</td>
<td>Analysis of accident factors</td>
<td>5. Consideration of action policy</td>
<td>6. Production of the measures to be implemented</td>
</tr>
<tr>
<td></td>
<td>Form of accident</td>
<td>Example of accident occurrence</td>
<td>Process of accident</td>
<td>Example of action policy</td>
<td>Example of measures to be implemented</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 6. Schematic layout of the Guideline

3-2 Use of the Guideline

This section explains how to use the Guideline for Road Safety Countermeasures. Prior to using the Guideline, the conditions of accident occurrence are so organized as in Figure 7 that the types of parties who caused the accidents, the directions of vehicles moving when the accidents occurred, the type of accident, or the severity of accident are clarified in Japan, where the conditions of accident occurrence are organized at locations where the actions are taken.

Based on this figure, designate the types of accident that intensively occur at the same locations or the forms of accident that particularly tend to become serious as the types of accident that are to be reduced by road safety countermeasures. Then, select the types of accident to be reduced by countermeasures set as
above from the types of accident, road segments with straight road or with intersection, and (1) the types of accident stated in the Guideline.

On the right of the column of any given type of accident selected are (2) the processes of accident occurrence assumed from the selected type of accident. Then, select (2) the process of accident occurrence among those listed that is considered appropriate by checking against the local status. When it is impossible to narrow candidates down to a single process, do not dare to narrow them down to a single one but select two or more.

Next, on the right of the selected process of accident occurrence are listed accident factors by the accident causer, which are (3-1) the mistakes made by the accident causers and (3-2) the road traffic environment that induced the mistakes of the accident causers. Therefore, select the assumed mistakes made by the accident causers (3-1) and the road traffic environment that induced the mistakes of the accident causers (3-2) by checking them against the local conditions. Since it is difficult to narrow down the factors only with (3-1) the mistakes of the accident causers, consider the road traffic environment that induced the mistakes of the accident causers to see if they match the local conditions and select the right factors.

Select the right action policy among the (4) action policies listed on the right of the selected mistakes (3-1) and the environment (3-2). Lastly, review countermeasures appropriate for implementation by referring to (5) the specific types of work used for road safety countermeasures.

When even young engineers with no sufficient experience or knowledge on road safety countermeasures go through this process, they can share rich experience and promote analysis of accident factors and formulation of appropriate countermeasures.

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Vehicles</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Motorcycles</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Cyclists</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Pedestrians</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Stopping vehicles</td>
<td></td>
</tr>
</tbody>
</table>

Figure 7. Diagram of accident occurrence

4. CONCLUSION

Although traffic accidents tend to decrease in number in Japan, it is still necessary to continue implementation of road safety countermeasures including the countermeasures by the road administrator, such as road improvement. The road administrator has so far taken various countermeasures and produced successful results to some extent. It is, however, still up to the experience and knowledge of engineers to realize implementation of effective countermeasures, and there are still locations where the effective results have not been obtained. Considering these circumstances, we established the Guideline that help share necessary experience and knowledge. It is expected that the road administrator use this guideline and effectively implement the road safety countermeasures.
**PAPER TITLE**  
UIRNet: The Italian National ITS Platform for integrated logistics

<table>
<thead>
<tr>
<th>AUTHOR</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Leonardo Domanico</td>
<td>Project Manager</td>
<td>TTS Italia</td>
<td>Italy</td>
</tr>
<tr>
<td><strong>CO-AUTHOR(S)</strong></td>
<td><strong>POSITION</strong></td>
<td><strong>ORGANIZATION</strong></td>
<td><strong>COUNTRY</strong></td>
</tr>
<tr>
<td>Olga Landolfi</td>
<td>Secretary General</td>
<td>TTS Italia</td>
<td>Italy</td>
</tr>
<tr>
<td><strong>CO-AUTHOR(S)</strong></td>
<td><strong>POSITION</strong></td>
<td><strong>ORGANIZATION</strong></td>
<td><strong>COUNTRY</strong></td>
</tr>
<tr>
<td>Rodolfo De Dominicis</td>
<td>President</td>
<td>UIRNet</td>
<td>Italy</td>
</tr>
<tr>
<td><strong>CO-AUTHOR(S)</strong></td>
<td><strong>POSITION</strong></td>
<td><strong>ORGANIZATION</strong></td>
<td><strong>COUNTRY</strong></td>
</tr>
<tr>
<td>Nicola Bassi</td>
<td>Project Manager</td>
<td>UIRNet</td>
<td>Italy</td>
</tr>
</tbody>
</table>

**E-MAIL**  
(leonardo.domanico@ttsitalia.it)

**KEYWORDS:**  
transport; information and communication technology; fleet management; dangerous goods; intelligent transport systems

**ABSTRACT:**  
Italy’s road network has one of the highest traffic densities in Europe. This is mainly due to the fact that road transport is by far the leading mode of transportation for both goods and passengers, even though Italy’s geographical features do not favour it compared to sea and rail transportation. This results in the Italian road system being characterized by frequent traffic congestions, safety and security issues, as well as environmental pollution.

In order to address these issues the Italian Ministry of Infrastructure and Transportation established UIRNet at the end of 2005. Its mission is to create a National ITS Platform for integrated logistics that could be successful in increasing efficiency, security and safety.

This paper will address the general ITS architecture of the aforementioned Platform, and outline the services and benefits that are planned for all the stakeholders involved in the logistics supply-chain.

UIRNet’s ITS Platform is based on the use of web-oriented data exchange solutions. It allows for full integration with external systems, can easily interface with other ITS platforms, and is fully accessible by mobile or fixed terminals and via internet. Furthermore, the Platform is accessible to public institutions, enabling them to have oversight.
1 Transport in Italy

Italy is one of the European countries with the highest density of internal traffic that is unevenly distributed across its transportation network that includes 290 ports, a rail network of 20,392 km, a road network (local roads) of approximately 254,700 km, a highway network of 6,668 km (of which 5,724.4 km toll road) and 44 airports.

In 2011 the volume of passenger-km for distances greater than 50 km [Source: National Accounts of Infrastructure and Transport, 2011-2012] was 885 billion, a clear indicator of a system with an intensive mobility rate and very dynamic production and exchange rates. Yet it also shows to the potential threat of saturation, and the serious limitations it would impose.

In fact, a heavy imbalance amongst modes of transportation makes the risk of collapse of the road transport system increasingly plausible: 91.53% of trips with routes greater than 50 km are carried out on road, while the remaining 8.13% is divided between railways (5.98%), air (1.89%) and waterways (0.44%).

In 2011 the total cargo traffic of national carriers with origin and destination within Italy, and travelling distances over 50 km was more than 200 billion tons-km/year, with 57.69% of the demand focused on the road, while the rest was distributed between rail / pipeline (14.71%) and inland waterways (27.08%). The percentage of freight transport by air is, on the other hand, irrelevant (0.52%). Rail and pipeline traffic, the share of international traffic carried out on national territory, are comprised in the data above.

This data clearly outlines the absolute preference for road transport over other modalities in Italy. Unfortunately, however, this preference is not matched by the infrastructure currently in place, which is not yet adequate, compared to the heavy demand. This shortfall results in extensive negative externalities, in terms of congestion, environmental pollution and safety.

Furthermore, the demand for road transport is not spread evenly on the territory: traffic flows are concentrated in a few critical road segments and nodes around major metropolitan and industrial areas. In fact the problem of traffic in urban areas has become very serious as we find 50% of the population, over 70% of production activities, and 60% of circulating vehicles all converging in one area. Moreover, in the past two decades we have seen a strong tendency for people to reside outside large cities, which has consequently increased the number of commuters, which in turn has increased traffic on the already stressed urban road network.

2 Freight transport in Italy

Logistics is a vital and crucial sector for every country’s national economy as it has real potential to give significant impetus to its growth. The current economic recession has highlighted just how much a country’s production process is tied to its logistic organization.

In fact a country’s ability to maximise profits on its production and be competitive on the global arena are strictly connected to its capacity to optimize the distribution process, and thus to its logistics sector.

The most recent statistics released by the Ministry of Infrastructure and Transport (2011-2012) highlight that the total quantity of transported goods in 2011 is around 200 billion ton-km, with a decrease of 16.2% compared to 2005. The available data confirms the absolute prevalence of road transport with the following value of ton-km and percentage:

- 114.736 Millions of ton-km (57.69%) via road (decreased compared to 2005)
- 53.852 Millions of ton-km (27.08%) via waterways (increased compared to 2005)
- 29.263 Millions of ton-km (14.71%) via railways and oil pipelines (decreased compared to 2005)
- 1.026 Millions of ton-km (0.52%) via airways, a very limited percentage (increased compared to 2005)

<table>
<thead>
<tr>
<th>Transport modes</th>
<th>Year 2005</th>
<th>Year 2008</th>
<th>Year 2011</th>
</tr>
</thead>
<tbody>
<tr>
<td>Waterways</td>
<td>46.928</td>
<td>47.081</td>
<td>53.852</td>
</tr>
<tr>
<td>Railways / Oil Pipeline</td>
<td>33.668</td>
<td>34.286</td>
<td>29.263</td>
</tr>
<tr>
<td>Road</td>
<td>155.872</td>
<td>136.952</td>
<td>114.736</td>
</tr>
<tr>
<td>Airways</td>
<td>982</td>
<td>999</td>
<td>1.026</td>
</tr>
<tr>
<td>Total</td>
<td>237.450</td>
<td>219.318</td>
<td>198.877</td>
</tr>
</tbody>
</table>

Table 1 – Modal shift overview Years 2005, 2008 and 2011 (Millions of ton-km)
Therefore road is the mode transport more used compared to other for the freight transport. The main reasons of this modal shift are the following:

- the average distance for freight transport in Italy is less than 300 km and therefore the road mode is the most efficient (main reason)
- the inefficiency of intermodal transport: there is a lack of a normative framework to achieve a better management of rail and maritime transport, despite the geographic shape of Italy that should favour the rail and maritime transport
- the disorganization of intermodal transport: some wagons/trailers often were missed in the past during the modal change and usually the modal shift is not so quick
- the trade association of road freight transport is very strong and obtains subsidies/contributions for toll and fuel costs.

The challenge in Italy now is to improve the transport system, favouring a more balanced modal to the detriment of the road and in favour of other modes; and UIRNet can play a key role in this process.

The routes and modalities for freight transportation are currently determined by the transporter himself based on his needs and preferences, instead of being based on what would be the most efficient and appropriate solution for the specific case (type of cargo, point of origin and destination, weather conditions, etc…).

This spontaneous organization of freight transport is the real system pathology and represents the greatest challenge in achieving what is called compatible development.

On the other hand, any form of liberalism and free market unequivocally necessitates the existence of a network that is:

✓ integrated and interactive with the territory
✓ homogeneous and without interruption
✓ capable of providing the conditions of real competitiveness between different stakeholders of the same network, guaranteeing the processes of modal interchange and reducing the damage caused by the “last mile”.

3 Intelligent Transport Systems & Solutions

In order to address the challenges of the increasing demand for transportation of goods, and to be in line with other European countries, the Italian Ministry of Infrastructure and Transportation has stated the necessity for Italy to rethink transportation within its territory.

The aim is to deal with transport as an “integrated system”, where information, management and control operate in synergy, optimizing the use of vehicles and infrastructure and of the existing logistics platforms in a multimodal manner. This is possible through Intelligent Transport Systems (ITS).

The ITS sector has been active in Italy since the ’80, but has seen significant development in the subsequent decade, in parallel with the growth of the sector in other major industrialized countries.

ITS are also essential for the optimizing of the freight transport. In fact adequate tools for transportation monitoring allow the improvement in warehouse management, stock planning, loading and unloading of
goods in the freight village. Moreover, transport managers can use information and data provided by the ITS to optimize and improve their services, e.g., by enabling their customers to track vehicles and schedule/manage the delivery/load.

In Italy, the Ministry of Infrastructure and Transportation promoted the development of a new ITS platform to overcome the above-mentioned challenges in the transport sector: the UIRNet platform.

4 The Italian legislative framework on ITS

On 6th August 2010, the European Commission published the ITS Directive 2010/40/UE adopted on the 7 July 2010. This Directive establishes a framework in support of the coordinated and coherent deployment and use of Intelligent Transport Systems (ITS) within the Union, in particular across the borders between the Member States, and sets out the general conditions necessary for that purpose. According to the ITS Directive 2010/40/UE, all Member States are requested to adopt this Directive and to implement an ITS National Action Plan.

Italy adopted the ITS Directive in 2012 and the ITS National Action Plan the last 12 February 2014. The Italian Plan includes four priority areas, reflecting those indicated by the ITS Directive 2010/40/UE:

1. Optimal use of road, traffic and travel data
2. Continuity of traffic and freight management ITS services
3. ITS road safety and security applications
4. Link the vehicle with the transport infrastructure.

The second priority area addresses the issues related to the achievement of safety conditions, efficiency, interoperability and continuity of ITS services for the management of traffic and transport, as well as those needed to stimulate inter-modality and co-modality in the European transport corridors and conurbations. The goal is to achieve integrated multimodal mobility, both for people and goods, to plan and manage the movement in an informed, personalized and seamless way from the point of origin to that of destination, using all the available modes efficiently and safely.

The development of integrated mobility services for both people and goods is based on: availability, access and set up of data and information that constitutes the key to enable such services; the management and organization of this data must be integrated platforms open and interoperable; same for payment systems integrated with the transport services. It is necessary that the different operators collecting and processing information on mobility, dialogue with those platforms.

The second priority area identifies the five priority actions needed to reach the objectives listed. Priority actions 1, 2 and 4 are particularly relevant to freight transport.

1) Priority action 1 aims at encouraging the creation of telematics platforms in the nodes of the distribution network that are consistent with UIRNet’s National Logistics Platform. This will allow the exchange of data, information and documentation between operators, which will improve, simplify and speed up all processes and administrative issues within the complex cycle of intermodal transport (road, rail and sea).

This will necessitate the promotion of an extensive information and training campaign aimed at the real users of telematics platforms, in order to facilitate their use and encourage the development of open ITS systems, which have to be interoperable with each other and with the National Platform for the Logistics UIRNet.

2) Priority action 2 aims at encouraging the use of ITS for the management of multimodal transport systems and logistics through open and interoperable platforms. Within the second priority action, the challenge is to implement ITS systems dedicated to logistics and freight transport, by stimulating and intensifying the intermodality and co-modality of transport both at national and international level, through 21 corridors recognized at European level.

ITS systems will have to be interoperable, standardized and will use UIRNet’s National Platform for the Logistics as reference point for road transport.

Furthermore, the continuity and the interoperability of the ITS services that facilitate the switch between road transportation to other modes in every node (ports, freight villages, stations and airports) will have to be guaranteed in terms of:

- release of information
- simplification of administrative proceedings
- fluidity of circulation in intermodal areas
- reduction of waiting times
- harmonization of interactions among the different actors involved through the National Logistics Platform, to manage:
  - freight transport information and the related e-documents,
tracking and tracing of vehicles transporting dangerous goods through RFID (Radio Frequency Identification), GPS (Global Positioning System) and EGNOS (European geostationary navigation overlay system)/Galileo,

- use to technologies to check status of both vehicles and goods, adopt standard protocols and open and interoperable ITS architectures for data exchange.

According to the specific characteristics of the Italian framework, special attention will be given to city logistics to control vehicle type and time of pick up and delivery of goods within the urban area.

3) Priority action 4 aims at guaranteeing the continuity of services on the national network and along borders. Within this priority action, the “interfacing” of national control systems of passenger and freight traffic, will be favoured at the European level to ensure the continuity of management services and information on the entire national network and along the borders, trans-borders collaborations with the other Member States will be established.

The logistic sector is crucial for Italy: this is underlined both by the ITS Action National Plan, as described above, and by the National Plan for Logistics 2011-2020: the Country could save up to 4 billion Euro by reducing the inefficiencies of the logistics sector.

The creation of an ITS National Platform for the Logistics is a primary step to improve the efficiency of the sector and support Italian economy.

5 The UIRNet Platform
In order to improve freight transport in Italy, the Italian Ministry of Infrastructures and Transportation established UIRNet at the end of 2005, with the mission of creating a National ITS Platform for integrated logistics. This platform has been implemented to make the national logistics system more efficient and secure.

It is very important to distinguish between different types of logistic platform, according to their complexity and operational integration:

- logistic areas
- multimodal platforms

Logistic areas involve integrated operations, with stock consolidation, local and re-directional activities. Cargo traffic is concentrated in these infrastructures, and they facilitate goods division and dispatching when switching to different modes of transport.

Multimodal platforms are logistic nodes connecting different modes of transport. These infrastructures are also known as hubs, which are usually linked to ports, to make the most of scale economies on international routes.

Regarding to the technological support to transport, some (proprietary) solutions are currently available on the market. Their services are generally based on the availability of Global Position Systems (GPS) positioning data, which enables managing fleets of trucks, and allows customers to monitor their goods during transportation via web interface, and so on.

Other ITS platforms enable the matching of suppliers and customers. Often these services are provided by different companies, which are not integrated into one unique platform. In fact, each one of these companies operates only in one market segment, and does not benefit, a comprehensive view of the status of the transport network (e.g., the status of traffic, the potential congestions of nodes, the visibility of the whole load and so on).

UIRNet’s integrated platform represents a new paradigm where services are integrated into an ITS platform. This is why we refer to UIRNet as an integrated logistics platform based on an extensive use of ITS solutions, so it does not correspond to a physical location (logistic area), as reflected by the definition given at the begin of this section.

UIRNet’s ITS Platform is based on the use of web-oriented data exchange solutions for full integration with other external systems. Furthermore, the Platform is accessible to the institutions, allowing them to directly oversee, and fully interface with other ITS platforms. Its functionalities are fully accessible by mobile or fixed terminals, and via Internet.

UIRNet’s ITS platform is consistent with the National and European rules for the development of telecommunications systems devoted to transportation. As described in the technical documentation UIRNet’s telecommunications system is designed to have a flexible and modular architecture able to support system scalability.

6 UIRNet System Architecture
The project focuses on the use of web-oriented data exchange solutions (Simple Object Access Protocol – SOAP and Extensible Markup Language – XML) and it allows the full integration with other external systems, such as legacy platforms for fleet management, ITS, etc. Furthermore, thanks to the system’s ability to easily integrate, it makes it open to the institutions and flexible to interface with other platforms. All the platform functions are fully accessible by mobile or fixed terminals, and via internet. The adopted communication infrastructure ensures a high level of security and protection against unauthorised access by a specific DMZ (De Militarized Zone). Disaster recovery policies are performed through data redundancy and the management of hazardous events. All the information transferred between on-board terminals, UIRNet Platform and nodes, transit through a private gateway ensuring both protections against interceptions and data integrity.

The platform architecture is composed by the following elements:

1. **Data centre**: it is the centre of the ITS platform where the hardware and applications for service delivery are set up. It has an extremely scalable architecture to guarantee the necessary upgrade (250,000 users are foreseen as the platform’s medium-long term capacity). The system takes advantage from a high reliability and security level. The data centre is characterised by:
   - high-speed connection through dedicated data lines
   - web oriented architecture to maximise the interoperability, the access capabilities to the platform and the fully integration with other platforms (logistics, institutions)
   - secure access and data storage due to a DMZ network segment.

2. **Situation room**: this is a monitoring and control centre where all data is collected, elaborated and visualised. The structure manages all anomalies, and emergencies that UIRNet users might incur. It acts as a common interface between the institutional bodies responsible for security and it also defines the code of conduct in case of emergency. The situation room is connected through a dedicated line with the data centre. It also supports different type of communications (mobile, fax, e-mail, web...).

3. **Communications gateway and on-board devices**: they respectively represent the mobile access point and the set of field terminals, or on board units. The communication gateway is connected to the data centre and represents its gateway through the mobile network. Thus, the communication gateway considers the data centre as a data convergence point (i.e., a centralised database) and an information dispatcher. On board devices are those that the UIRNet platform user can adopt to trace vehicles, monitor goods (especially dangerous goods) and manage logistics activities. There are simple on board units, and more advanced ones, that can carry out more complex operations.

4. **Contact centre**: it is the main reference point for both subscribed users, and prospective users (i.e., registration management and accounting). The contact centre is connected to the data centre with a dedicated line. Its communications are aimed directly to the users/subscribers (voice communications) or transit via data centre. The contact centre supports different types of voice and data communications (mobile, fax, e-mail, web).

UIRNet’s platform is divided in two main functional areas:

1. **UIRNet Alert area**, which includes the info-mobility services addressed to assist the freight route, up to its delivery. It implements the information system, data exchange, fleet control and monitoring tools, load and dangerous goods management systems.

2. **UIRNet web services**, which support other services like e.g. the meeting between supply and demand and the workflow management of the entire logistics processes (including document management and transport operations management, etc.)

Macro Services are built over a combination of basic functions and modules. They are briefly described in the following paragraph. Every module is interconnected to the others via a specific middleware:

1. **UIRNet portal (PUR)**: it is the web access point to information and services offered by UIRNet. The web portal contains useful information dedicated to the logistics community and potential UIRNet subscribers; there is also a private area for subscribers. Every user only has access to the functionalities he is authorised to by its company manager (user profiling).

2. **Alert management system (ALR)**: it manages the information about vehicles and road and traffic conditions. The system manages the complete list of notifications and pre-alarms that can be collected by the service centre (based on automatic alarms) or manually sent by ‘push’ mode. More precisely, ALR manages the following different types of alarms/messages:
   - automatic notifications about transported goods (time and physical parameters like e.g. the goods’ temperature)
1. Automatic alarms in case of route deviations (corridor monitoring)
2. Automatic alarms concerning the crossing of pre-determined areas (geo fencing)
3. Information about traffic and weather conditions
4. Alarms and information manually sent by users via Contact centre.

3. **Geo-referencing system (GIS):** it is an application of a geographic information system (GIS). It is accessible via web e and it allows the visualisation of the vehicles’ position, traffic conditions (represented by icons) and other events (congestion, road work…). It also allows the representation of any other geo referenced objects.

4. **Middleware integration with third party applications:** it is the communication module dedicated to the integration of applications with legacy systems of institutions and logistics operators. The integration middleware is the first level interface between the platform and the external world, represented for example by institutional systems, public and private logistics platforms, ITS and fleet management systems. Due to the employed technologies, this system can support data exchange with future platforms by standard service oriented architecture protocols (SOA).

5. **Data ware-housing system (DWH) and reporting:** it refers to the application that allows to periodically extract the aggregate data about traffic movement and logistics.

6. **CRM system:** this module performs data collection, management and consolidation of information related to the interactions between the UIRNet Platform and its users. CRM system includes a set of facilities for the front-office (external relations) and for the back-office. It allows data analysis, data measurement and performance estimation.

7. **Mobile communications services and billing (CMB):** it includes application platforms to support communications with mobile networks. It includes voice communications solutions (SCV) and data communications solutions (SCD).

8. **Web services components:**
   - Booking system (PRT): the system allows transport operators to book different services (load and unload premises). It also able to send confirmation via web or mobile device.
   - Document management (DOC): the system of document management allows to obtain an integrated and automated exchange of documents between the stakeholders engaged in the transport operations to guarantee a high level of security and confidentiality of the information.

---

**Figure 3 - UIRNet system architecture**

---

7 **UIRNet web portal**

UIRNet’s web portal is the most direct access point to the services and for the registration of new users. It makes the promotion and dissemination of information regarding the offered services simply and efficiently. Moreover, it enables interaction and communication with users thanks to a variety of services. The services of the portal include:
Horizontal services: these are highly interactive services available on the portal and aimed at favouring communications and interaction among UIRNet’s registered users, logistics operators and other stakeholders.

Application services, or vertical services: this group includes services offered by a unique user interface. They allow consultation and/or upgrade of the database (transactions) directly via internet-extranet. Such services imply the realization of specific software modules that give access to the information in a simple way, allowing also other data operations. The system allows users to plan a trip (referred to as ‘mission’) and its route, while the duration can be automatically estimated. The users can see a complete data regarding their past and current activities, and can quickly set up data analysis and reports. In addition, the use of freight taxi systems becomes essential for growing businesses.

8 Conclusions and way forward
The paper presented a brief analysis of the transport sector in Italy and UIRNet’s new platform. The platform allows single players to benefit from advanced solutions based on real-time information concerning traffic and road conditions, vehicles and goods. The platform also constitutes an effective support in the management of documents and in the transportation of dangerous goods.

The adoption of highly innovative communications solutions combined with satellite localisation, tracking & tracing, and data exchange solutions allows us to maximise both the efficiency and the quality of the offered services, especially in the transport sector. It also plays a crucial role in obtaining a competitive advantage in terms of a noticeable differentiation, which is however limited to a specific market niche, the logistics sector. By contrast, UIRNet’s platform gives to the entire world of logistics a new strategic opportunity to improve the efficiency and security of the entire supply chain. This has a significantly positive impact on the national economy in terms of increased competitiveness, security, environmental benefits, as well as opportunities for internationalisation. UIRNet’s initiative will greatly improve the efficiency and safety of Italy’s transport system: the system is tailored to address the territory’s characteristics and each stakeholder’s needs, optimizing each process, maximising assets and minimizing negative externalities.

Further developments of the project will allow the pursuing of significant strategic objectives as the project grows and more systems integrate. This is expected to have a positive impact in terms of:

- Increasing competitiveness through the increase of intermodal, the development of freight villages in terms of services and market positioning, as well as the integration with other logistics systems at national and international level.
- Increasing safety as UIRNet’s initiative can be used to monitor and control the transport of goods, especially dangerous goods.
- Internationalisation through the integration with European and Asian dry ports, and even integration with any other logistics projects worldwide.
- Decrease of environmental impact through the development of intermodal transport, the management of goods and of hazardous waste.

Considering the opportunities created by UIRNet, it is expected to become a pillar in the field of transportation logistics.

References
(2) Italian ITS decree of the 1th February 2013 promoted by the Ministry of Transport in collaboration with the Interior Ministry and Research Ministry on the diffusion of ITS in Italy - Published on Italian Official Journal the 26th March 2013 (13 pages)
(3) Italian ITS Action Plan adopted and published by the Ministry of Transport the 12th February 2014 (43 pages)
The Plans to Support the Operations of Express Buses via JOBAN Expressway as countermeasures against Great East Japan Earthquake

Integrated Mobility & ITS

Toll Service Section, Operation Department
Tohoku Regional Head Office, East Nippon Expressway Co., Ltd
Japan

- Operation Department
Tohoku Regional Head Office, East Nippon Expressway Co., Ltd
Japan

h.shishinai.aa@e-nexco.co.jp

KEYWORDS:
Express buses
Promoting of the operation of expressway buses
Installation of bus stops

ABSTRACT:

Great East Japan Earthquake occurred on March 11 in 2011 when the construction of section between Joban-Tomioka and Yamamoto in Joban Expressway which runs through the coastal area of Fukushima pref. was mostly completed. Besides, the construction was interrupted by the accident at the Tokyo Electric Power Fukushima Daiichi nuclear power plant that is close to the section.

In 2012, East Nippon Expressway Co., Ltd. (NEXCO-East) has resumed construction after decontaminating radioactive wastes, aiming to complete in the spring of 2015. On the other hand, the railway and national route that run in the same region have also suffered a great deal of damage and the operation of public traffic has been suspended.

The expressway network in Tohoku region has been expanding rapidly. Thus, a large number of express buses run instead of railways.
NEXCO-East, taking charge of construction, operation and maintenance of expressway, has performed the following measures to support the operations of express buses:
- Installation of bus stops which annex/adjoin expressways
- Installation of parking lots with bus stops for park&ride-system
- Discount of the expressway tolls for express buses

Therefore, we are planning to install some bus-facilities for express buses. The report introduces the following articles:
- The countermeasures for construction of JOBAN expressway after the earthquake disaster
- The situation of express buses in Tohoku region, and our various measures to support them
- The plans to facilitate the operation of express buses after full opening of JOBAN Expressway, and safety measures to radiation risk
The Plans to Support the Operations of Express Buses via JOBAN Expressway as countermeasures against Great East Japan Earthquake

Haruka SHISHINAI, Toll Service Section, Operation Department, Tohoku Regional Head Office, East Nippon Expressway Co., Ltd., Japan
Koichi ABE, Tohoku Regional Head Office, East Nippon Expressway Co., Ltd., Japan
Hiroyuki IKEDA, Operation Department, Tohoku Regional Head Office, East Nippon Expressway Co., Ltd., Japan

OVERVIEW

East Nippon Expressway Co., Ltd. (NEXCO East Japan) is now constructing the section between Joban-Tomioka and Yamamoto Interchange on Joban Expressway. Although the construction was suspended for a time due to the accident of Tokyo Electric Power Company’s Fukushima Daiichi Nuclear Power Plant caused by Great East Japan Earthquake on March 11th, 2011, the construction was resumed again and we are aiming at opening in spring 2015. Damaged railways and national roads in the same area are not under operation yet.

Joban Expressway runs through the Tohoku Region, where population is decreasing due to declining birthrate and aging population. However local railroads supported the public transportation in the region in the past, they are now ruined and very inconvenient (Abe et al. 2000). On the other hand, the network of expressways developed a lot and many express buses are under operation. This is because the convenience and economic efficiency of express buses which have been highly valued by people who need the transportation between regions and the services increased with more ridership.

So the reopening of Joban Expressway and operation of express buses are supposed to reinforce the transportation network in the south Tohoku Region. Above all, in the stricken area where the prospect of operation of railways and national roads is still dim, the expressway is highly expected as a help for rehabilitation.

This paper introduces the effort of NEXCO East Japan to promote express bus services. At the same time, it tells the scheme of NEXCO East Japan to realize the operation of convenient express bus, aiming the full operation of Joban Expressway.

1. The state and expected effect of entire opening of Joban Expressway

1–1 The state of Joban Expressway

Joban Expressway is a national expressway which stretches from the Kanto Region, including the metropolitan area, to Miyagi Prefecture via Fukushima Prefecture; the total length is about 310 km. At present, sections between Misato Junction and Joban-Tomioka Interchange, between Minami-Soma and Soma Interchange, and between Yamamoto and Watari Interchange, totaling about 250 km, are under operation. Sections between Joban-Tomioka and Minami-Soma Interchange and between Soma and Yamamoto Interchange, totaling 60km, are under construction. (Figure1&2)

It was March 11th, 2011 at 02:46 p.m. when an earthquake of magnitude 9.0 occurred in the Pacific off the coast of the Tohoku Region. It was the biggest earthquake observed in Japan. The earthquake caused cracks on the road, differences in level, bridge slippages, damages in bridge bearing and cutting of optical fiber cables at 4000 points of expressways in the Tohoku Region. Furthermore, tsunami hit the coastline. Tollgates were flooded, rubbles rushed and expressways were blocked.

NEXCO East Japan finished emergency restoration work within 20 hours right after the earthquake and enabled emergency vehicles to run on the road. Full restoration work, started after emergency work, completed in 2012. A section of Joban Expressway (between Hirono and Joban-Tomioka Interchange), which is very close to Fukushima Daiichi Nuclear Power Plant, was force to closed for a long period but NEXCO started the full restoration work in March 2012. With the coordination with the Ministry of Environment for decontamination work, the Expressway was finally reopened in February 22nd, 2014.

About sections under construction, the roads were partially damaged by the earthquake. The Fukushima Daiichi Nuclear Plant, about 5km far from these sections was also damaged and the regions around it were contaminated by radiation. Therefore the construction had been suspended, but it resumed from April in 2014 controlling the exposure dose of workers and NEXCO East Japan aims full-opening of Joban Expressway in the spring of 2015.
Figure.1 Expressway network formed by the completion of Joban Expressway
Figure 2 The sections under construction, Joban Expressway

1 – 2 The expected effect of entire opening of Joban Expressway

Tohoku Expressway, which is another route that links the Kanto and Tohoku Region, was place in service until Sendai in 1975. After the full-opening of Joban Expressway, these 2 lines will make a mutually complementary form. For example, Joban Expressway runs through coastal area, where snow is less than the mountain area where Tohoku Expressway runs. When Tohoku Expressway is closed temporarily in winter, Joban Expressway secures the transportation and the redundancy of transportation network increases.

2. The circumstance of expressway bus

2 – 1 The circumstance of transportation in the Tohoku Region

The development of expressway network which shows arborescent pattern, expressway buses have become an important transportation tool instead of railways, which tend to provide fewer numbers of service and more expensive fare. Then, today various expressway buses are under operation and cities in the Tohoku Region are connected with other cities in the region or the metropolitan area by expressway buses.

According to the data 2012 which was published by Tohoku District Transport Bureau of the Minister of Land, Infrastructure, Transport and Tourism, total ridership of expressway busses which arrive and depart in the Tohoku Region was 9 million. Half of them travel between cities of 6 prefectures in the Tohoku Region. In addition, 80 % of these buses in the region arrive at and depart from Sendai City, Miyagi Prefecture. Sendai City has the biggest
population in the Tohoku Region. Other than that, buses go to the metropolitan area and other regions from the Tohoku Region count for about 10% of the total and the other buses are operated within each prefecture.

Seeing the breakdown of ridership between Miyagi or Fukushima Prefecture and the metropolitan area, where Joban Expressway runs through, about 200,000 people in Miyagi Prefecture and about 500,000 people in Fukushima Prefecture ride on buses for the metropolitan area. Compared with other prefectures, more people in these 2 prefectures use expressway buses.

Furthermore, ridership between Miyagi and Fukushima Prefectures was about 1 million but in the previous year, 2011, when the earthquake occurred, ridership was about 800,000. The figure shows that needs for expressway buses increased with the suspension of railways due to the nuclear disaster and the advancement of reconstruction.

2 – 2 Approaches to promote the operation of expressway buses

Unlike railroad companies which operate trains, NEXCO East Japan does not operate expressway buses directly. Then NEXCO East Japan has carried out various kinds of measures to promote the operation of expressway bus since the time of Japan Highway Public Corporation, the predecessor of NEXCO, in cooperation with bus companies and local municipalities. The following are major approaches:

a. Toll discount system only for expressway buses

From the viewpoint to improve the convenience of local people which brought by developing public transportation system, expressway bus toll is discounted on certain conditions.

For example, when an expressway bus stops at a bus stop outside the tollgate, the toll is calculated based on the through movement. In Japan, expressway toll for over than 100km is decreased for a specific rate. However, without the discount system for express buses, their running distance is cut short before reaching 100km even though they once go out the tollgate only to stop at a bus stop.

Other than that, 30 % discount of toll is adopted for buses which stop at 75% or more bus stops on expressways.

b. Development of bus stop on expressway

To set up bus stop, local municipality, bus companies, and NEXCO East Japan set up a conference and discuss the plan. After that, the municipality applies for the bus stop installation to the national government then the matter is examined and judged. Once the installation is approved, the municipality carries out the construction of bus stop or platform and the shed of bus stop is treated as the object for exclusive use by the municipality. (Photo1 and 2)

Photo1: Ono Bus Stop, Joban Expressway (at interchange)

Photo2: Iwaki-Nakoso Bus Stop, Joban Expressway (at interchange)
c. Development of park and ride facilities

NEXCO East Japan cooperated with local municipalities and installed park and ride facilities next to some bus stops. In rural areas, public transportation is inconvenient to take expressway buses. Especially in early morning or night, it is almost impossible to use public transportation to access to expressway bus stop. Therefore people are usually dropped off or picked up at bus terminals. Under such circumstance, we see more cases where parking lot for expressway bus commuters is developed to enable them to access by their cars for enhanced convenience.

One of the cases is the bus stop set at Iwaki-Chuo Interchange, Joban Expressway (photo 3 and 4). Iwaki City, where Joban Expressway runs through, free parking lot (for about 150 cars) was installed by the expressway bus company for customers. However, with the increase of customers, development of additional parking lot was an urgent task. In 1999, NEXCO East Japan (Japan Highway Public Corporation), Iwaki City and the Bus Association launched a joint venture to install a new parking lot (for 157 cars) under the viaduct near Iwaki-Chuo Interchange. It is exclusive for the transfer to expressway buses. Furthermore, rest room and waiting room are installed along with the parking lot and enhanced convenience for customers who use the bus stop at the interchange was realized.

3. Full-opening of Joban Expressway and route change plan of expressway buses

3 – 1 The intention of expressway bus companies for bus operation and the response from NEXCO East Japan

We conducted hearing survey with 6 bus companies about the operation plan after the Full-opening of Joban Expressway. They are 2 bus companies which operate bus between Sendai and Iwaki via Tohoku Expressway and 4 companies which operate bus between Sendai and Soma and between Sendai and Minami-Soma via national roads. (Figure 2)

On hearing survey, NEXCO East Japan emphasized the opening plan of Joban Expressway and the characteristics of route as follows. For example, in the section between Sendai and Iwaki-Chuo Interchange, if Joban Expressway is used instead of Tohoku Expressway, the distance is shorter by about 45km, the time is shorter by about 20 min. and the toll is less expensive by more than 10%.

Based on such information disclosure, Company A and B, which operate bus between Sendai and Minami-Soma, showed a big expectation for the full-opening of Joban Expressway and Company A said that it scheduled to increase the number of expressway bus service.

On the other hand, many bus companies expressed the concerns about the response to traffic jam or long stopping due to problems in high dosage area, securing the detour route when road is closed, and the impression which users hold to pass through the high dosage area.

To cope with such concerns, NEXCO East Japan is planning to decrease dosage not more than 50mSv/year, which is the target value, by installing thick pavement of 35cm on contaminated expressway. This thickness is based on the result of test decontamination work in the restricted area of Joban Expressway, conducted by Ministry of the
<table>
<thead>
<tr>
<th>Company</th>
<th>Location of the headquarters</th>
<th>Route and number of service (a day)</th>
<th>Route, etc.</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Minami-Soma City, Fukushima Prefecture</td>
<td>Between Sendai and Minami-Soma, 10 round-trip operation</td>
<td>Shinchi Town Office</td>
</tr>
<tr>
<td>B</td>
<td>Minami-Soma City, Fukushima Prefecture</td>
<td>Between Sendai and Minami-Soma, 6 round-trip operation</td>
<td>Shinchi Town Office, operation with C</td>
</tr>
<tr>
<td>C</td>
<td>Sendai City, Miyagi Prefecture</td>
<td>Between Sendai and Minami-Soma, 6 round-trip operation</td>
<td>Shinchi Town Office, operation with B</td>
</tr>
<tr>
<td>D</td>
<td>Sendai City, Miyagi Prefecture</td>
<td>Between Sendai and Iwaki 3 round-trip operation</td>
<td>Tohoku Expressway, operation with E</td>
</tr>
<tr>
<td>E</td>
<td>Iwaki City, Fukushima Prefecture</td>
<td>Between Sendai and Iwaki 5 round-trip operation</td>
<td>Tohoku Expressway, operation with D</td>
</tr>
</tbody>
</table>

Figure 2: The companies we conducted hearing survey

Environment on March in 2012. In addition, NEXCO East Japan is scheduling to set up monitoring posts and provide the dosage information in real time to relieve concern among people.

3 – 2 Development plan of bus stop cooperated with local municipalities

Shinchi Town, Fukushima Prefecture, is developing the bus stop next to the Shinchi Interchange on Joban Expressway, with the aim of completion in 2016.

The expressway bus stop at Shinchi Town Office is for local commuters who ride on the expressway bus which runs from Sendai City to Soma City or Minami-Soma City. Around Shinchi Interchange, there are many public housing estates and factories. Once Joban Expressway is opened, the time to Sendai City is shortened and timelines increases. It will be more convenient for commuters, especially who live and work around Shinchi Interchange.

So Shinchi Town has decided to develop a bus stop next to Shinchi Interchange. (Figure 3) At the same time, the town plans to make a parking lot at the bus stop for convenience of commuters who have been using Shinchi Town Office bus stop currently. The parking lot allows park and ride.

When the bus stop is developed, it is expected that not only people around Shinchi Town but also people in Marumori Town or Kakuda City in Miyagi Prefecture, both are next to Shinchi Town, would come to the bus stop.
4. Toward the area rehabilitation

With the full-opening of Joban Expressway, it is expected that the cooperation of bus companies and municipalities would create a convenient expressway bus service. NEXCO East Japan has ties with bus companies and municipalities. So we are aiming to play the role of collecting necessary information and arranging with related parties to realize the development of bus stop and realization of bus operation by three parties.

Now, Shinchi Town is developing the bus stop plan. NEXCO East Japan provides information and suggestions based on the measures of bus stop development which NEXCO East Japan had conducted. By doing so, NEXCO East Japan supports the bus stop development project of municipality.

Joban Expressway is sometimes called “the life road to support the rehabilitation” by people in surrounding area. They suffered the Great East Japan Earthquake. NEXCO East Japan supports the expressway bus operation which supports the local transportation as well as the development of Joban Expressway for people to use safely.

We believe that helping suffered people and local municipalities could contribute to the rehabilitation in the Tohoku Region entirely.

REFERENCES

PAPER TITLE
(90 Characters Max)

A Study on Risk Evaluation Method for Bridge Asset Management

TRACK

AUTHOR
(Capitalize Family Name)

POSITION

ORGANIZATION

COUNTRY

CHOI, Hyun-Ho

Team Manager

Organizing Committee of XXVth World Road Congress

KOREA (Republic of)

CO-AUTHOR(S)
(Capitalize Family Name)

POSITION

ORGANIZATION

COUNTRY

PARK, Kyung-Hoon

Senior Researcher

Korea Institute of Civil Engineering and Building Technology

KOREA (Republic of)

E-MAIL
(for correspondence)

choihyunho@msn.com

KEYWORDS:
Risk Analysis, Asset Management, Bridge, Performance Measures, Level of Service

ABSTRACT:
Risk-based performance evaluation method and procedure for asset management is suggested. To apply this, condition assessment, performance measuring, assessment of failure modes and risks, evaluation/selection of strategy options, and implementation of optimum solutions are included.

First, guideline of risk evaluation level according to bridge inventory information is suggested. And second, performance measure such as noise near bridges, load carrying capacity, level of condition, vulnerability against disaster, difficulty of inspection and diagnosis, etc are classified. Then, evaluation method for quantitative/qualitative performance measure including weighting value is proposed. Also, based on the design and maintenance related standards, evaluation method of risk analysis for bridge asset management is suggested and basic research is carried out for applicable method of risk-based asset management. In this study, for the applicability of suggested procedure and appropriateness of guideline of risk evaluation level, operating 4 bridges in Korea are selected and case study is carried out.

Moreover, it is concluded that the proposed applicable method of risk-based asset management will provide a solution to contribute the development of systematical asset management for optimal decision making and prototype asset management system.
A Study on Risk Evaluation Method for Bridge Asset Management

Choi, Hyun-ho¹; Park, Kyung-Hoon²

¹Organizing Committee of XXVth World Road Congress, Seoul, Korea (Republic of)
²Korea Institute of Civil Engineering and Building Technology, Gyeonggi, Korea (Republic of)

Email for correspondence: choihyunho@msn.com

1. INTRODUCTION

The importance of bridge maintenance is highlighted when discussing infrastructure, because of the loss of life and property, and the social and economic effects caused by the damage to or collapse of bridges. Bridge construction in South Korea has increased a rapid growth, along with the expansion of its road network in support of economic development. Of total bridges built, approximately 77.6% and 37.3% were constructed within the past 20 and 10 years respectively (MLTM 2012). It is expected that the demands of bridge maintenance will burgeon as the bridges deteriorate after long usage periods, and that the time and cost of such works will be concentrated over a single specific period. Hence, asset management is essential for the sake of securing budgetary requirements for optimized maintenance; making budget savings through preventive maintenance; and preventing the concentration of budget expenditure, etc. (Park, Kyung-Hoon, et al., 2009)

Related studies on the asset management of bridges and roads have been the subject of recent attentions in Korea. However, detailed asset management procedure and methods for each type of infrastructure have not been set up yet, especially including risk. Thus, this study, as basic research for establishing risk-based bridge asset management, proposes an asset management procedure and methodology incorporating risks, which is practical and applicable in general. Bridge Inventory (BI) and Performance Measures (PM) in the field of asset management refers to an index that is used to quantitatively or qualitatively evaluate risk levels of the asset; quantitative/qualitative risk evaluation standard and methods based on the BI and the PM. Also, for the applicability of suggested procedure and appropriateness of guideline of risk evaluation, 4 bridges in use are selected and case study is carried out.

2. RELATED STUDIES ON ASSET MANAGEMENT

The International Infrastructure Management Manual (IIMM) (INGENIUM & IPWEA 2006) issued by the National Asset Management Steering Committee (NAMS) is one of the most representative research and case studies regarding infrastructure asset management ever undertaken. The manual shows some risk assessment case studies. The basic asset management framework suggested by the IIMM consists of the acquisition of accurate information about assets; the selection of service levels; assessment of demand levels; assessment of financial standing and available budgets; preparation of asset management plans; and the acquisition of revised information about assets. The upgraded procedure, which includes risk assessment, adds condition evaluation and performance measurement; assessment of damage types and risk levels; the evaluation and selection of responses; and the implementation of optimum alternatives. In this manual, the rough results of incidents were uniformly converted into expenses in order to rank and access risk levels. Based on the asset management framework of the IIMM, this study considered the most prevalent asset management procedure, and proposes an evaluation process considering risk levels for the practical bridge asset management.

Michael Baker Jr., in “Risk Management Strategy for Bridges and Structures (2009),” suggested a method for combining index of road classification called the Business Plan Network (BPN) and the categorizing condition level. Then, based on this method, a risk index is suggested for asset management. The suggested risk assessment would be classified into two methods. The first method calculates risk scores taking account of both direct functions (e.g. load carrying capacity, erosion) and indirect functions (e.g. current status of bridge substructure, bypass). The second method focuses only on structural defects according to the condition level of structures. Thus, Baker’s study (2009) extended risk assessment to incorporate indirect functions for bridge asset management. Before Baker’s study, usually risk assessment was performed to be limited to assess simple direct functions (e.g. condition level, load carrying capacity).
In this study, risk factors are similarly identified by affecting the risk degree and level of bridges directly and indirectly as following the reference (Michael Baker Jr. 2009). Based on this identification, an applicable risk evaluation procedure and method considering the risk both of BIs and PMs is suggested.

3. RISK-BASED ASSET MANAGEMENT FOR BRIDGES

In previous studies, risks are scarcely considered in the process of asset management (Park, Kyung-Hoon et al., 2009) and the service level evaluation (Sun, Jong-Wan et al., 2011). To support more rational and practical decision-making on bridge maintenance, it is necessary to assess appropriate risk levels according to service levels and in response to find optimized alternatives for maintenance, rather than be limited to simply setting up service levels and assessing demands.

As shown in Figure 1, the procedure of risk-based asset management for bridges is proposed. Firstly, accurate inventories of each bridge’s characteristics must be investigated in order to rationally carry out asset management. The managing entity establishes maintenance policies and plan, and then the appropriate service level is set up. To quantify the evaluation of each service level, standards of PM levels are decided. Then, a risk assessment is performed by evaluating the risk and calculating the risk scores according to the BI evaluation and the previously decided PM standards. Finally, after deciding prioritization, alternatives are implemented.

![Risk-based Asset Management Procedure for Bridges](image)

Figure 1. Risk-based Asset Management Procedure for Bridges

3.1 RISK IDENTIFICATION FOR BRIDGE ASSET MANAGEMENT

Risk classification is necessary in order to recognize the possibility of any inherent bridge risk and to avoid overlooking any incidents. Inherent risk may be classified into two types: risk caused by location or other inherent conditions until bridge construction, and risk influenced by environment while in use. First, in order to induce appropriate inherent risk through the BI, we referred to Baker’s study (2009) and defined six risk factors as listed below. This list consists of essential information managed for institutional purposes, especially in domestic characteristics. These 6 inventories can be used as inherent risks for all bridges:

BI 1. Bridge grade
BI 2. Bridge age
BI 3. Dimensions and special bridge types (e.g. suspension bridge)
BI 4. Location of overpass road or railway
BI 5. Flooded area
BI 6. Allowance of overloaded vehicles
Sun, Jong-Wan et al. (2011) suggested 21 PMs in order to quantify the service level of a bridge. The PMs define items that can be used to examine most incidents that may occur while in use, such as noise levels around bridges; energy efficiency; efficient connection with roads; appropriateness in bridge management budgets; appropriateness in tolls; load carrying capacity in performance; condition levels; disaster vulnerability; possibility of safety inspection and assessment; provision of traffic information; appropriate response to emergencies; and receiving of complaints, etc. However, there are some limits in the aforementioned PMs in order to suggest methodologies and to explain cases incorporating risk levels for bridges on roads; for example, appropriate or not to be used as a representative performance index for measuring specific bridges to use as an overall performance index for road infrastructure that is at a higher level; that are related to user-oriented services or not; and difficulties to prove rationally when trying to quantify qualitative factors.

In this regard, this study selected only five indexes that can be practically used for quantification through risk assessment on each PM in order to improve credibility and utilize it for deciding prioritization. Evaluation and combination of the PMs which are eligible for actual quantification according to a risk assessment method is the key to bridge asset management incorporating risk levels:

PM 1. Noise levels around a bridge
PM 2. Load carrying capacity in performance
PM 3. Condition levels
PM 4. Disaster vulnerability
PM 5. Difficulty of inspection and diagnosis

3.2 RISK ASSESSMENT METHOD FOR BRIDGE ASSET MANAGEMENT

Usually, rational risk assessment requires an assessment of consequence, i.e. calculation of failure probability, by considering repair costs, decrease in income, shortfall of service levels, casualties, threats against assets, and continuous shortage of demands. In a case where calculation of failure probability is almost impossible, common methods are to classify risks based on qualitative and quantitative terms. In qualitative approach, qualitative risk levels (e.g. A–E) are classified, and in quantitative approach, the probability for possible quantitative risks (e.g. 0.02) are calculated and then, also rank them (Seo & Choi 2008; Baker 2009). In this study, an evaluation method for qualitative risks was applied in order to assess the risk levels of BI. If objective classified standards and information is existed regarding the aforementioned BI (BI 1–6), risk levels are accordingly classified. For some risks, risks are divided with respect to environmental conditions of the specific bridge’s location. Risk levels were graded on a scale of 1 to 5, where 5 represents the highest risk and 1 the lowest.

A risk matrix method was used to perform PM quantification. The risk matrix is evaluated by the occurrence and magnitude of risks (INGENIUM & IPWEA 2006). When assessing various risk levels in asset management for big infrastructure, it takes too much time to measure actual risk levels in general, and in some cases an absolute lack of data makes reasonable risk evaluation impossible. Hence, considering practical applicability, the customized risk evaluation standard was established and applied as rationally and feasibly as possible after limiting it to the aforementioned PMs (PM 1–5). In this study, it is impossible to define the correlation between risk levels related to the BI and PMs, therefore, their correlation presumed as mutual independence. Moreover, taking into account the importance of each assessment step, the weighted values are included in this methodology to measure the final risk levels.

3.3 RISK EVALUATION ACCORDING TO THE BI

For risk assessment according to the suggested BI (BI 1–6), the evaluation standard and score of judgments were shown in Table 1. The evaluation and judgment standards may be appropriately altered for application depending on the asset management entities or the policies and purposes of bridge maintenance. This study suggests the evaluation standards considering the bridge design and conditions. For the BI 1, it was clearly classified as following the Korean bridge design standards (MLTM 2010). For the BI 2 and 3, through statistical analysis, cumulative probability distribution calculated from bridge status data was incorporated so that appropriate ranges are suggested. For the BI 4 to 6, inclusion of risk levels was considered mainly in order to suggest the judgment condition, but subdivision may be possible once classification of more detailed current status is quantified. In order to determine the weighed value according to the BI, expert surveys and interviews were conducted, and the Analytic Hierarchy Process (AHP) was used to determine the weighed value based on the collected data.
Table 1. Risk evaluation standard according to the BI

<table>
<thead>
<tr>
<th></th>
<th>Evaluation Standard</th>
<th>Score of Judgments</th>
<th>Weights</th>
</tr>
</thead>
<tbody>
<tr>
<td>BI 1</td>
<td>Bridge grade</td>
<td>1st grade - 1, 2nd grade - 3, 3rd grade - 5</td>
<td>0.07</td>
</tr>
<tr>
<td>BI 2</td>
<td>Bridge age</td>
<td>30 years older - 5, 20-30 year old - 3, under 20 years - 1</td>
<td>0.19</td>
</tr>
<tr>
<td>BI 3</td>
<td>Maximum span</td>
<td>100m over - 5, 50m-100m - 3, less 50m - 1</td>
<td>0.13</td>
</tr>
<tr>
<td>BI 4</td>
<td>overpass rail or road</td>
<td>Yes - 5, No - 1</td>
<td>0.30</td>
</tr>
<tr>
<td>BI 5</td>
<td>Overpass river or stream</td>
<td>Yes - 5, No - 1</td>
<td>0.08</td>
</tr>
<tr>
<td>BI 6</td>
<td>Allow overloaded truck</td>
<td>Yes - 5, No - 1</td>
<td>0.23</td>
</tr>
</tbody>
</table>

3.4 RISK EVALUATION ACCORDING TO THE PM

An assessment of quantitative standards of road noise found that general residents judge approximately 40dB to be the minimum noise level made by passing vehicles, and 60 - 70dB to be the threshold level beyond which the risk level is heightened. As shown in Table 2, five categories were classified into the most common type of risk matrix, according to the scope of risk level with regards to the noise levels around bridges. Regarding the possibility of noise from bridges, five categories were classified as shown in Table 2, taking into account the regional characteristics of the bridge’s location and traffic volume, which is the major cause of noise. Risk levels were calculated based on the evaluation of possibility and magnitude by connecting occurrence probability and risk seriousness in order to conclude a final risk level according to the matrix, as shown in Table 3. For example, if a noise level was recorded as 65dB, the risk level on Table 2 was “Level 3,” and if the bridge was located in a suburb rural area, the occurrence was “Low.” Consequently, the final risk level of the bridge, according to Table 3, was “Medium.”

Based on the risk level evaluated according to the aforementioned procedure, noise risk of bridges is represented as one of five categories as shown in Table 4, and similar categories of risk levels are defined for PMs (PM 2-5) as well. Any managing entity, unless the PM-based risk level reached the highest level (Level E), will need to take note of the final results of risk assessment, and then prioritize and establish responsive measures for the improvement of each performance. Five items from PM 2 to PM 5, were evaluated in this way. First, for the “PM 2 Load carrying capacity,” the result directly calculated by a certified load carrying capacity evaluation method was used for risk assessment in order to quantify a load carrying capacity that represents safety of the load. Also, the “PM 3 Condition levels” of each bridge was calculated based on the status inspection result of each subsidiary material and span. The status evaluation of subsidiary materials includes compositional subsidiary materials of the bridge, and concrete carbonation and chloride. Differential application is used to consider the importance of each subsidiary material so that the current status can be calculated in the most rational way. In the case of the “PM 4 Disaster vulnerability,” disasters that may only occur to bridges were mostly limited to earthquakes and floods. In order to quantify vulnerability to such disasters, the seismic design, vertical clearance and horizontal clearance of bridges over streams or rivers were considered when classifying risk levels. Other disasters that may possibly occur in South Korea, such as extreme wind and tsunami, were excluded due to comparatively low damage and occurrences. For the “PM 5 Difficulty of inspection and diagnosis,” the easiness of bridge inspection by using the equipment and temporal scaffolding for routine regular inspection and non-destructive test such as neutralization, steel corrosion, surface strength, and excessive deflection, were examined.

Table 2. Risk degree standards for Possibility and Magnitude of Noise

<table>
<thead>
<tr>
<th></th>
<th>Magnitude</th>
<th>Possibility</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level 1</td>
<td>Under 50dB, Low risk</td>
<td>&quot;Very High&quot; Locate downtown urban or ADT over 50,000 vehicle/day</td>
</tr>
<tr>
<td>Level 2</td>
<td>50dB - 60dB, Low risk</td>
<td>&quot;High&quot; Locate suburb urban or ADT over 25,000 vehicle/day</td>
</tr>
<tr>
<td>Level 3</td>
<td>60dB - 70dB, Normal risk</td>
<td>&quot;Medium&quot; Locate downtown rural or ADT over 10,000 vehicle/day</td>
</tr>
<tr>
<td>Level 4</td>
<td>70dB - 90dB, High risk</td>
<td>&quot;Low&quot; Locate suburb rural or ADT over 5,000 vehicle/day</td>
</tr>
<tr>
<td>Level 5</td>
<td>Over 90dB, Very high risk</td>
<td>&quot;Very Low&quot; Locate remote rural or ADT less 5,000 vehicle/day</td>
</tr>
</tbody>
</table>

Table 3. Risk Matrix for Noise

<table>
<thead>
<tr>
<th>Magnitude</th>
</tr>
</thead>
</table>
Table 4. Risk category for Noise

<table>
<thead>
<tr>
<th>Category</th>
<th>Risk</th>
<th>Rank</th>
<th>Definition</th>
<th>Response Strategy</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Very High</td>
<td>E</td>
<td>Risk rank is defined as “Very High”</td>
<td>Need urgent response</td>
</tr>
<tr>
<td>2</td>
<td>High</td>
<td>D</td>
<td>Risk rank is defined as “High”</td>
<td>Need Cautious check</td>
</tr>
<tr>
<td>3</td>
<td>Medium</td>
<td>C</td>
<td>Risk rank is defined as “Medium”</td>
<td>Need normal check</td>
</tr>
<tr>
<td>4</td>
<td>Low</td>
<td>B</td>
<td>Risk rank is defined as “Low”</td>
<td>Negligible</td>
</tr>
<tr>
<td>5</td>
<td>Very Low</td>
<td>A</td>
<td>Risk rank is defined as “Very Low”</td>
<td>Negligible</td>
</tr>
</tbody>
</table>

4. CASE STUDY OF RISK-BASED ASSET MANAGEMENT

The risk-based bridge asset management was applied for 4 actual bridges in usage to approve its applicability and appropriateness. In order to effectively show the result of this study, 4 bridges located on national highways were assessed, as shown in Figure 2. Their BI and dimensions were obtained through the Bridge Management System (BMS) of the managing entities, and on-site investigations were performed to assess characteristics caused by the locations and environments of the bridges.

(a) Bridge A  (b) Bridge B  (c) Bridge C  (d) Bridge D

Figure 2. Example Bridges for case study

Table 5. Risk evaluation results according to the BI

<table>
<thead>
<tr>
<th>BI</th>
<th>Evaluation Standard</th>
<th>Bridge A</th>
<th>Bridge B</th>
<th>Bridge C</th>
<th>Bridge D</th>
</tr>
</thead>
<tbody>
<tr>
<td>BI 1</td>
<td>Bridge grade</td>
<td>2nd grade</td>
<td>1st grade</td>
<td>2nd grade</td>
<td>1st grade</td>
</tr>
<tr>
<td>BI 2</td>
<td>Bridge age</td>
<td>43 year</td>
<td>21 year</td>
<td>45 year</td>
<td>7 year</td>
</tr>
<tr>
<td>BI 3</td>
<td>Maximum span</td>
<td>Under 50m</td>
<td>Under 50m</td>
<td>Under 50m</td>
<td>Under 50m</td>
</tr>
<tr>
<td>BI 4</td>
<td>overpass rail or road</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>BI 5</td>
<td>Overpass river or stream</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>BI 6</td>
<td>Allow overloaded truck</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>No</td>
</tr>
</tbody>
</table>

Table 5 shows the result of risk assessment conducted based on the BI. According to the judgment condition and weighted value according to the BI shown in Table 1, the result of its BI was scored as shown in Table 5, and the final risk score incorporating the weighed value was demonstrated as shown in Table 6. Table 6 shows the risk score for each bridge as follows: Bridge A – 2.22, Bridge B – 1.96, Bridge C – 1.90, Bridge D – 0.81. When simply considering the risk level shown with the BI, the result means that Bridge A with the highest score is with the highest risk, whereas Bridge D is relatively safer.
Next, the risk matrix method was applied to evaluation of PMs. Each 5 PMs were evaluated by utilizing the information collected from the BMS, on-site investigation, and inspection data as shown in Table 7. The risk level and occurrence considering the characteristics of each PM were evaluated, and the risk score according to the PM was similarly calculated using the risk matrix, as already shown in Table 3.

Table 6. Risk scores according to the BI

<table>
<thead>
<tr>
<th>Bridge A</th>
<th>Bridge B</th>
<th>Bridge C</th>
<th>Bridge D</th>
<th>Bridge A</th>
<th>Bridge B</th>
<th>Bridge C</th>
<th>Bridge D</th>
</tr>
</thead>
<tbody>
<tr>
<td>BI 1</td>
<td>0.07</td>
<td>3</td>
<td>1</td>
<td>3</td>
<td>1</td>
<td>0.21</td>
<td>0.07</td>
</tr>
<tr>
<td>BI 2</td>
<td>0.19</td>
<td>5</td>
<td>3</td>
<td>5</td>
<td>0</td>
<td>0.95</td>
<td>0.57</td>
</tr>
<tr>
<td>BI 3</td>
<td>0.13</td>
<td>1</td>
<td>3</td>
<td>1</td>
<td>1</td>
<td>0.13</td>
<td>0.39</td>
</tr>
<tr>
<td>BI 4</td>
<td>0.30</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>BI 5</td>
<td>0.08</td>
<td>5</td>
<td>5</td>
<td>1</td>
<td>1</td>
<td>0.4</td>
<td>0.4</td>
</tr>
<tr>
<td>BI 6</td>
<td>0.23</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0.23</td>
<td>0.23</td>
</tr>
<tr>
<td>Total</td>
<td>1.00</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2.22</td>
<td>1.96</td>
</tr>
</tbody>
</table>

Table 7. Risk evaluation results according to the PM

<table>
<thead>
<tr>
<th>Bridge A</th>
<th>Bridge B</th>
<th>Bridge C</th>
<th>Bridge D</th>
</tr>
</thead>
<tbody>
<tr>
<td>(PM 1) Noise levels around a bridge</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ADT</td>
<td>31,934</td>
<td>4,313</td>
<td>4,313</td>
</tr>
<tr>
<td>Location</td>
<td>Downtown urban</td>
<td>Remote rural</td>
<td>Remote rural</td>
</tr>
<tr>
<td>dB</td>
<td>69.4</td>
<td>22.4</td>
<td>22.4</td>
</tr>
<tr>
<td>Risk level</td>
<td>Level 3</td>
<td>Level 1</td>
<td>Level 1</td>
</tr>
<tr>
<td>Possibility</td>
<td>High</td>
<td>Very Low</td>
<td>Very Low</td>
</tr>
<tr>
<td>Risk evaluation</td>
<td>High</td>
<td>Very Low</td>
<td>Very Low</td>
</tr>
<tr>
<td>Risk Score</td>
<td>4</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>(PM 2) Load carrying capacity in performance</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Load carrying rate</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td>Risk level</td>
<td>Level 2</td>
<td>Level 2</td>
<td>Level 2</td>
</tr>
<tr>
<td>Possibility</td>
<td>High</td>
<td>Very Low</td>
<td>Very Low</td>
</tr>
<tr>
<td>Risk evaluation</td>
<td>Medium</td>
<td>Very Low</td>
<td>Very Low</td>
</tr>
<tr>
<td>Risk Score</td>
<td>3</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>(PM 3) Condition levels</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slab</td>
<td>b</td>
<td>b</td>
<td>c</td>
</tr>
<tr>
<td>Girder</td>
<td>a</td>
<td>b</td>
<td>x</td>
</tr>
<tr>
<td>Pavement</td>
<td>b</td>
<td>c</td>
<td>c</td>
</tr>
<tr>
<td>Abutment/Pier</td>
<td>b</td>
<td>b</td>
<td>b</td>
</tr>
<tr>
<td>Shoe</td>
<td>x</td>
<td>a</td>
<td>b</td>
</tr>
<tr>
<td>Joint</td>
<td>b</td>
<td>c</td>
<td>c</td>
</tr>
<tr>
<td>Curb/Handrail</td>
<td>a</td>
<td>a</td>
<td>a</td>
</tr>
<tr>
<td>drainage facility</td>
<td>b</td>
<td>b</td>
<td>a</td>
</tr>
<tr>
<td>Average condition</td>
<td>B</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>Risk level</td>
<td>Level 2</td>
<td>Level 2</td>
<td>Level 3</td>
</tr>
<tr>
<td>Possibility</td>
<td>High</td>
<td>Very Low</td>
<td>Very Low</td>
</tr>
<tr>
<td>Risk evaluation</td>
<td>Medium</td>
<td>Very Low</td>
<td>Low</td>
</tr>
<tr>
<td>Risk Score</td>
<td>3</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>(PM 4) Disaster vulnerability</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seismic design</td>
<td>No</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>horizontal clearance risk</td>
<td>0</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>vertical clearance risk</td>
<td>0</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>Risk level</td>
<td>Level 5</td>
<td>Level 5</td>
<td>Level 5</td>
</tr>
<tr>
<td>Possibility</td>
<td>High</td>
<td>Very Low</td>
<td>Very Low</td>
</tr>
<tr>
<td>Risk evaluation</td>
<td>Very High</td>
<td>Medium</td>
<td>Medium</td>
</tr>
<tr>
<td>Risk Score</td>
<td>5</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>(PM 5) Difficulty of inspection and diagnosis</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
The risk score according to the PM as shown in Table 7 was calculated in the form of a total weight which added the weighted value per item, and Table 8 shows the final results. As similarly deciding the weighted value of the BI, the herein weighted value of risk level per PM was measured by taking into account 2nd trial of expert surveys and interviews using the Delphi method. Table 8 shows the risk score for each bridge as follows: Bridge A – 3.75, Bridge B – 1.44, Bridge C – 1.67, Bridge D – 3.21. The result of PM-based assessment - which may be said to be more qualitative than the BI - concludes that Bridge A and B have a high level of risks, while Bridge B and C have a low level of risks. The final risk scores, as shown in Table 9, were calculated as follows: Bridge A – 2.99, Bridge B – 1.70, Bridge C – 1.79, Bridge D – 2.01. It means that a consideration of both the BI and the PM concluded that Bridge A has the highest risk level. The bridge which was concluded to have the highest risk turned out to also have the highest risk in the PM-based assessment. However, Bridge C, which was assessed to be relatively risky in the BI-based assessment, appeared to be relatively safer than Bridge A or Bridge D in the PM-based assessment. Rather, Bridge D ranked no. 2 in the risk level when considering both the BI and the PM.

Table 8. Risk Scores according to the PM

<table>
<thead>
<tr>
<th>Weights</th>
<th>Bridge A</th>
<th>Bridge B</th>
<th>Bridge C</th>
<th>Bridge D</th>
<th>Risk Scores × Weights</th>
</tr>
</thead>
<tbody>
<tr>
<td>PM 1</td>
<td>0.21</td>
<td>4</td>
<td>1</td>
<td>1</td>
<td>0.84</td>
</tr>
<tr>
<td>PM 2</td>
<td>0.24</td>
<td>3</td>
<td>1</td>
<td>1</td>
<td>0.72</td>
</tr>
<tr>
<td>PM 3</td>
<td>0.23</td>
<td>3</td>
<td>1</td>
<td>2</td>
<td>0.69</td>
</tr>
<tr>
<td>PM 4</td>
<td>0.22</td>
<td>5</td>
<td>3</td>
<td>3</td>
<td>1.1</td>
</tr>
<tr>
<td>PM 5</td>
<td>0.10</td>
<td>4</td>
<td>1</td>
<td>1</td>
<td>0.4</td>
</tr>
<tr>
<td>Total</td>
<td>1.00</td>
<td>19</td>
<td>17</td>
<td>14</td>
<td>3.75</td>
</tr>
</tbody>
</table>

Table 9. Results of Risk Assessment

<table>
<thead>
<tr>
<th>Weights</th>
<th>Bridge A</th>
<th>Bridge B</th>
<th>Bridge C</th>
<th>Bridge D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Risk scores (BI)</td>
<td>0.5</td>
<td>2.22</td>
<td>1.96</td>
<td>1.90</td>
</tr>
<tr>
<td>Risk scores (PM)</td>
<td>0.5</td>
<td>3.75</td>
<td>1.44</td>
<td>1.67</td>
</tr>
<tr>
<td>Risks</td>
<td>2.99</td>
<td>1.70</td>
<td>1.79</td>
<td>2.01</td>
</tr>
</tbody>
</table>

5. CONCLUSION AND FUTURE STUDY

In this study, risk-based evaluation method and procedure for asset management is suggested. To apply this, condition assessment and performance measuring for maintenance strategy are included. This study also suggested methods both using the BI and PM. For application, case studies were also conducted to verify the applicability. It is judged that such a method can help to efficiently consider the quantitative and qualitative risk levels inherent in bridges and provide rational assessment results.

This study suggested a risk level of standards according to the BI. Also, the study selected the PMs that require consideration of risk levels, including noise levels around a bridge, load carrying capacity in performance, condition levels, disaster vulnerability, and difficulty of inspection and diagnosis. It also suggested the PM standards so that qualitative evaluation decided the weighted values to determine the relative level of importance. As the judgment standard was established based on design and maintenance standards, it is expected that such a method could be utilized as the actual risk evaluation standard and method.
In order to verify the applicability and appropriateness of suggested assessment and judgment standard, case study was performed. For this, 4 bridges in current usage were selected. The BI data were collected from the managing entities, and also the necessary information was obtained from additional on-site investigations. As a results, calculated final risks in this case study shows that the results of risk assessment are more reasonable in case of both considering the BI and the PM simultaneously rather than that of risk assessment which simply considered from the BI.

It is concluded that the proposed applicable method of risk-based asset management will provide a solution to contribute the development of systematical asset management for optimal decision making and prototype asset management system.

ACKNOWLEDGEMENTS

This work is a part of a research project supported by Korea Institute of Civil Engineering and Building Technology. The authors wish to express their gratitude for the financial support.

REFERENCES


PAPER TITLE | Mode Separation with a Purpose – A Traffic Management Approach to Bring Order in Dhaka’s Chronic Traffic Problems
--- | ---
TRACK | Integrated Mobility & ITS, Managing Mobility in Megacities

AUTHOR | POSITION | ORGANIZATION | COUNTRY
--- | --- | --- | ---
Nisar Ahmed | Program Coordinator | MTC | USA

E-MAIL | nisar.u.ahmed@gmail.com

KEYWORDS: Dhaka, chaotic traffic, mobility, mode separation, one-way traffic, transportation demand management (TDM), systematic traffic flow

ABSTRACT:

Dhaka is arguably the most densely populated urban area in the world with a density of 44,000/km² with less than 10% land allocation for transportation. Traffic in Dhaka grinds to a virtual ‘stand still’ for more than seven hours daily. All types of motorized and non-motorized traffic compete for the scarce roadways. Various TDM measures implemented by the government have largely failed to put a dent on city’s severe traffic congestion.

While government needs to seriously think of ways to bring the city population under control, there are practical approaches that can be implemented immediately in order to harness the full capacity of the transportation network. Approaches may include physical separation of non-motorized, low occupancy motorized, and public transit modes of transportation, introduction of highly efficient mass transit with isolated stops, and configuration of most local streets for one-way traffic. In addition, various TDM tools such as carpooling, biking/walking, effective traveler information, etc. should be aggressively promoted. Mode separation and one-way traffic can bring about desperately needed order in Dhaka’s chaotic traffic, hence providing the base for effective utilization of the network.

Dhaka cannot build its way out of the severe traffic problem it faces. Innovative approaches ingrained in country’s socio-economic conditions can provide more efficient mobility.
Mode Separation with a Purpose – A Traffic Management Approach to Bring Order in Dhaka’s Chronic Traffic Problems

Nisar Ahmed

1Metropolitan Transportation Commission, Oakland, CA, USA
Email for correspondence: nisar.u ahmed@gmail.com

1 A HEART STOPPING PROBLEM

If Dhaka, the capital of Bangladesh, is the heart of the country’s economic, social, and cultural activities, the traffic congestion in that city is the heart stopping problem for the entire country. Dhaka is arguably the most densely populated urban area (amongst urban areas with population of 500,000 or more) and megacity (megacity defined as the urban area with population of more than 10,000,000) in the world (Cox 2012, Demographia 2014) with a population density of 44000/km². Dhaka cannot afford to allocate anywhere near the standard practice of 25% of land necessary for transportation infrastructure for a planned urban development. Traffic congestion in a city of more than 15 million people crammed in a small land area of 347 km² (Cox 2012) and even scarce transportation space is a no brainer. Numerous studies have revealed the woes of Dhaka traffic congestion. An article in CNN Travel website (Bennett 2012) calls Dhaka traffic the worst in the world. For Dhaka, the definition of rush hour has been turned upside down because people can actually rush a bit during the time of least traffic. During peak hours, traffic in Dhaka grinds to a virtual ‘stand still’ for more than seven hours daily. A 2012 research study (Mahmud et al. 2012) provides an extensive list of causes and impacts of traffic congestion in Dhaka. The study also identifies a series of short, medium, and long term remedial measures to improve traffic conditions in Dhaka.

As shown in the Mahmud et al. and many other literatures, the causes of traffic congestion in Dhaka are numerous including overpopulation, unauthorized and overwhelming number of rickshaws (a manually pedaled tri-cycle), illegal parking, encroachment of sidewalks by vendors and illegal takeover of public spaces, inadequate or faulty traffic signals, inadequate roadway space in both length and width, lack of enforcement and adherence to traffic rules, unplanned roadway excavation, too many modes of transportation with varying levels of automation, and most importantly, inaction of traffic management experts to help the situation. These and many other problems can be broadly classified into transportation supply and demand management categories. Table 1 below shows the most pressing supply and demand factors contributing in Dhaka’s traffic congestion.

<table>
<thead>
<tr>
<th>SUPPLY SIDE</th>
<th>DEMAND SIDE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lack of urban planning</td>
<td>Over population creating huge mobility demand</td>
</tr>
<tr>
<td>Disproportionate (low) land allocation for transportation network</td>
<td>Unplanned growth in car ownership by the growing middle class</td>
</tr>
<tr>
<td>Incompatible mixed mode disrupting each other</td>
<td>Lack of efficient multi-modal transportation</td>
</tr>
<tr>
<td>Inefficient traffic signal management</td>
<td>Inefficient public transport</td>
</tr>
<tr>
<td>Lack of traffic law enforcement</td>
<td>Inadequate footpath, bike lanes, parking</td>
</tr>
</tbody>
</table>

Like many other cities in the developing world, Dhaka has not grown in a planned way. Even if some level of development guidelines were adopted, those weren’t fully implemented due to lack of enforcement. Had there been adequate planning and proper implementation, there would have been much greater share of land allocation for transportation network, there would have been more orderly operation of mixed mode transportation, and there would have been sufficient and synchronized traffic signals in operation reducing the need for traffic police on the streets. Capacity of Dhaka’s scarce transportation network already cannot support the unbridled growth of private cars, disorderly mixed use development without proper traffic impact studies and lawlessness on the streets. Dhaka’s current mobility demand of 25 million trips a day (DTCA
2014) is only likely to get worse as the time goes and population of the city continues to climb. With Dhaka’s population projected to surge beyond 22 million by the year 2020, mobility in Dhaka is fast approaching to a complete halt unless a comprehensive mobility plan is adopted and implemented immediately.

Impacts of Dhaka’s traffic congestion is severe - to say the least - and long lasting. If Dhaka is the heart of Bangladesh’s economic activities, then the transportation networks within the city and connecting it to the rest of the country is the network of artery of that economy. Disruptive mobility severely disrupts the natural economic growth. Economic impacts of disruptive mobility in Dhaka are manifold including, loss of productivity and output, rise in physical and mental healthcare costs, degradation of quality of life, irreparable damage to the environment, and many more. Efficient mobility through the transportation network of Dhaka is critical for the economic development of the country. Bangladesh, as a rising economy, cannot afford to have this strangling mobility problem holding down the otherwise thriving economy.

2 AN AILING HEART CAN BE RESUSCITATED OR REPLACED

With proper care and planned efforts a weak heart can be brought to health only before it stops working and completely shuts down the entire body and the brain. Efficient mobility of people and goods is the key to country’s economic vitality. As the traffic congestion reduces so increases the mobility. There is no silver bullet for reducing traffic congestion. Both supply and demand have to be managed in a coordinated fashion. Many thinkers of Dhaka’s traffic congestion problem have come up with a number of good ideas to improve the situation. A few are impractical but most are attainable with proper planning and careful execution. Table 2 below lists ideas for traffic congestion relief categorized into supply and demand side approaches.

Table 2. Ideas for alleviating traffic congestion in Dhaka

<table>
<thead>
<tr>
<th>SUPPLY SIDE</th>
<th>DEMAND SIDE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Implement one-way streets and grade separation</td>
<td>Relocate large establishments and international</td>
</tr>
<tr>
<td></td>
<td>trading activities outside of the city</td>
</tr>
<tr>
<td>Develop adequate pedestrian facilities</td>
<td>Reduce private establishments and remove</td>
</tr>
<tr>
<td></td>
<td>non-mechanized vehicles including rickshaw</td>
</tr>
<tr>
<td>Improve public transportation with rapid transit</td>
<td>Stagger business hours by activity type</td>
</tr>
<tr>
<td>Provide dedicated bike lanes</td>
<td>Control city population growth</td>
</tr>
<tr>
<td>Build flyovers, expressways</td>
<td>Manage parking through high fees</td>
</tr>
<tr>
<td>Modernize traffic signals and other controls</td>
<td>Implement car free days</td>
</tr>
<tr>
<td>Build more road and network connectivity</td>
<td>Implement and encourage shared mobility</td>
</tr>
<tr>
<td>Remove illegal encroachments</td>
<td>Fully enforce traffic laws</td>
</tr>
</tbody>
</table>

Transportation Demand Management (TDM) has been a key strategy for traffic congestion alleviation for quite some time in the developed world with mixed success. Irrespective of the past successes, TDM continues to be touted as one of the key congestion mitigation strategies as the world tries to reduce the carbon footprint of transportation. Encouraging alternative transportation modes such as walking, biking, and public transit along with transit oriented development (TOD), containment of urban sprawl, imposing higher cost of driving through increased parking fees, gasoline taxes, and roadway tolls, and ridesharing are key TDM strategies being promoted and implemented in various parts of the world. From time to time various strategies, as listed on the demand side in table 2, to manage travel demand in Dhaka have been tried with limited success not worth mentioning. These strategies included relocation of government offices in other cities only to see them come back, banning rickshaws on major city thoroughfares resulting in mobility inefficiency for a large segment of the population and creating higher congestion on other streets, late opening/closing of shopping centers and staggered weekly days of operation for shopping centers having negligible impact on traffic, etc. Car free day idea is likely to generate huge resistance, hence government won’t be interested. Even if it is implemented it will only mask the traffic congestion problem for some days and might create higher traffic demand following days. Proper enforcement of traffic laws is a given. Without enforcement no policy can be successful. On the demand side, Bangladesh government needs to implement a sustained long-term strategy to control population growth in Dhaka and for shared mobility. All past attempts
by the Bangladesh government to reduce traffic demand in Dhaka were primarily unsuccessful because those were very reactive approaches and not as part of a comprehensive mobility plan.

Success of TDM strategies largely depends on human behavior changes. Desired behavioral changes in terms of transportation choices people make can be achieved through sustained implementation of policies that can influence such changes. Policies need to support long-term mobility goals through most efficient management of existing infrastructure in addition to building new infrastructures as necessary, supporting a low carbon transportation system. Only building expensive new infrastructure cannot solve traffic congestion problem (Mann 2014) unless people are motivated by clearly demonstrated economic benefits in order to adopt TDM strategies in their transportation choices. When a country gains economic prosperity its demand for mobility and transportation grows. A good example is China. Demand for automobile in China has grown manifold over the last few decades with the increased buying capacity of prosperous Chinese middle class. Chinese cities are now faced with unprecedented traffic congestion and air pollution generated from the vehicular tailpipe emission. Even though China has the land and money to build new transportation infrastructure, they are fully aware that this problem cannot be solved only through building new infrastructure. China is now aggressively promoting TDM strategies in combination with the development of new transportation infrastructure to satisfy the mobility needs of the future.

Most of the supply side ideas in table 2 can be achieved through better management of existing infrastructure. Building more arterials and connecting roads are almost impossible in Dhaka due to very dense developments that must be demolished in order to build new roads. A comprehensive and coordinated long-term mobility plan for Dhaka must address both supply and demand side issues supported by appropriate policies. Below are a set of supply and demand side ideas that can be implemented to help bring order to the chaotic traffic in Dhaka providing the foundation for long-term mobility planning and policy implementation.

3 SUPPLY SIDE CONSIDERATIONS

Dhaka is heavily developed and the population density is exasperated by the conversion of one/two story single unit houses into high-rise apartment complexes. There is no land space left to expand existing or build new transportation infrastructure except for building multi-layer expensive vertical infrastructure. Even though the allocation of land for transportation within the existing built-out city is not feasible, it is conceivable to support higher proportion of the traffic throughput on the existing network if the capacity is fully utilized through better management and organization of traffic. Money spent in establishing order in traffic would likely have higher return on investment (ROI) than expensive new infrastructure. Major thoroughfares in Dhaka are reasonably wide, but not utilized efficiently. For efficient utilization and better management of given roadway space following supply side strategies should be considered. These strategies are not expected to solve Dhaka’s congestion problem completely, but would be a good set of first steps to bring order in the traffic chaos that exists today, a necessary prerequisite for future mobility planning.

1. **Separation of modes**: The single most detrimental cause for Dhaka’s traffic congestion is the mixed mode traffic vying for space on the roadways. To some, rickshaw is the menace, and to others it is the private car that is the culprit. The fact is that both these and other modes that need roadway space are not going to go away; in fact, they are all needed and can co-exist through better management. Figure 1 illustrates how much the population is depended on rickshaw. While the demand for cars and rickshaws must be identified and managed, it is not necessary to put them at odds with each other. What is needed is to ensure that various modes can operate smoothly without disrupting each other. That goal can be achieved by segregating each travel direction of all major streets into dedicated pathways for modes categorized into three – non-motorized rickshaws and bicycles, low occupancy motorized vehicles, and high occupancy buses. When buses, cars, and rickshaws are allowed to travel conflict free in their dedicated space they all have the potential to travel at their natural speed and not get bogged down by jam created by other modes resulting in better flow. Even if one mode slows down it won’t affect the other modes.

   In a mode segregated roadway, buses may travel on inner lane with at ground or elevated pullouts/ramps for stops. Inner lane operation and isolated stops will prevent busses from making illegal stops. Low occupancy vehicles should travel in the middle lane(s) with dedicated access/egress and turns
allowed only at designated locations. This will allow cars to travel at their natural flow and prevent them from making illegal stops and street parking. Rickshaw is part of Bangladesh culture and identity and should be preserved as long as possible. Rickshaw has its own industry that provides employment for millions of people and provides transportation for millions more. Unless a viable transportation mode for the masses is introduced as replacement and employment for the displaced workforce can be found, rickshaw should not be removed. In order to better manage rickshaw traffic the left-most part of the pavement should be dedicated for them. Two lanes in each direction for non-motorized traffic will help rickshaws and bicycles travel separate from buses and autos. A dedicated fourth lane in between bus and car lanes when space permits, or the bus/car lane should be designated for emergency vehicles such as ambulances, fire engines, and to clear stranded vehicles blocking roadway due to accidents or malfunction. Also the sidewalks must be adequate, properly designed, and encroachment free to ensure smooth pedestrian traffic. Figure 2 provides an illustration, created with http://streetmix.net, of the proposed mode separated thoroughfare.

Figure 1. Rickshaw is the indispensable mode in Dhaka

![Figure 1](image1)

Figure 2. A schematic of proposed mode separated Dhaka street

![Figure 2](image2)

2. Efficient and desirable high occupancy transportation: A double-decker or articulated bus can carry many more travelers than the cars occupying the same roadway space. From purely a transportation

---

space utilization perspective, cars are the most inefficient because they occupy more space per traveler than any other modes (Litman 2013). Figure 3 demonstrates roadway space necessary for 72 people travelling by bicycles vs. cars vs. a bus. This picture was commissioned by the planning department of the city of Munster, Germany. Most often than not, cars are not fully occupied. On the other hand, a bus (regular, double-decker, or articulated) can carry large number of travelers resulting in far more efficient use of the transportation space.

A public bus system can be much cheaper solution than subway/metro if operated efficiently maintaining a desired level of service and with easy access/egress through conflict free stops. An example of highly successful public bus system is the Curitiba Bus Rapid Transit (BRT) system in Curitiba, Brazil (Goodman et al. 2007). An automated fare collection system can generate pilferage free revenue that might be enough to pay for the operating cost. Even though car traffic has grown exponentially in Dhaka in recent years a large portion (>40%) of the mobility needs of Dhaka is met by the existing bus system (Mahmud et al. 2012). A more efficient bus service operating in a dedicated lane will likely increase the share of this mode as a result of some rickshaw passengers switching over to the bus mode, resulting in higher revenue for the bus operators in a system that already generates a healthy profit. Higher revenue creates the potential for increased competition among bus operators, which is likely to further increase the level of service. A more efficient and comfortable bus system might also attract some car travelers back to public transit. A dedicated bus lane can later be converted into a metro rail track. A public transit system operating on a dedicated lane without all the amenities of a true BRT can be coined as “Bus-only Lane Transit” or, in short, BLT.

Figure 3. Road space occupied by 72 people on bicycles vs. cars vs. a bus

3. Better traffic flow on local streets: Unlike megacities in developed world traffic congestion in Dhaka starts from the local streets. Due to increased car ownership, narrow local streets that were not designed for mixed mode bi-directional traffic often gets clogged from car traffic. Traffic flow on local streets can be smoothed by implementing one-way traffic with some connector streets providing bi-directional flow between one-way streets. One-way streets will increase the capacity and flow on local streets by allocating the full pavement width to traffic in the same direction. Where street width permits, one-way streets may provide separate lanes for motorized and non-motorized vehicles. For connectivity to major thoroughfares two parallel one-way streets can be used – one for access and the other for egress. These one-way connectors to major roadways should be designed to allow for motorized and non-motorized vehicles getting on/off the thoroughfares conflict free. Also, large buses and trucks may be restricted on certain local streets.

4 DEMAND SIDE CONSIDERATIONS

Demand for mobility will continue to grow as the population of Dhaka grows and economic activities continue to thrive. Smart management of existing infrastructure and travel demand, and building new facilities only when absolutely necessary should be the approach. In order to better manage demand for mobility approaches should be developed based on a few principles. These principles include most efficient use of available transportation space, dissemination of traveler information to help travelers make the right travel choice, and transformation towards environmentally sustainable transportation. Based on these principles, following mobility demand management measures should be considered for adoption.

1. **Encourage biking and walking:** All sidewalks and crosswalks (at grade, elevated, and underground) must be encroachment free and safe. This is easier said than done. It requires strong political will and strict enforcement. Until a safe walking environment is provided, foot traffic is likely to spill over to the pavement disrupting vehicular traffic and creating traffic congestion. Bicycle traffic should be combined with rickshaws in the dedicated non-motorized outer lane. It is possible that bicycle, walking, and rickshaw modes combined can support 45% - 50% of mobility needs of Dhaka dwellers given that rickshaws alone currently supports 38% of mobility needs (Mahmud et al. 2012) and bicycling is gaining popularity among young travelers. Meeting close to 50% transportation demand with zero pollution modes would be exemplary. In order to achieve that goal an allocation of about 40% of the travelway width that includes the sidewalk for these non-motorized modes would be justified.

2. **Encourage high occupancy in private cars:** Presently, 8% of mobility needs in Dhaka are met by private cars and another 11% by taxis and three-wheelers, popularly known as CNGs. The name CNG is derived from the abbreviation of its operating fuel, Compressed Natural Gas (CNG). Combined together, these low occupancy modes support less than 20% of mobility needs, but they occupy more than 30% of travelway width. Allocation of no more than 30% width or a single lane for these modes would provide them more than their fair share. In parallel, government should promote and help people share rides. If the allocated road space feels inadequate to car travelers it might motivate them to consider ridesharing or carsharing. In some parts of the world higher occupancy in private cars are incentivized through special carpool lanes, tax deductions, and other monetary incentives. Such incentives are likely to be abused in Bangladesh due to lack of enforcement and corruption and will not produce desired results. Instead, car owners should be encouraged to share available seats in their cars with fellow travelers in order to reduce congestion on the dedicated car lane.

3. **Facilitate effective traveler information:** Information of the prevailing travel conditions can help travelers make the right choice and help reduce congestion. Knowledge of current traffic bottlenecks and status of public transit service can help travelers decide whether to take an alternate driving route, use public transit, or bike/walk. Information systems can help travelers form ridesharing pools with other travelers having similar trip patterns. Government can either develop these information resources or encourage private sector to develop them through grants, competitions, soft-term bank loans, etc. Effective traveler information can help reduce congestion.

4. **Develop policy for environmentally sustainable transportation:** An environmentally sustainable mobility policy should prioritize efficient use of non-motorized and public transportation modes and gradual shift towards pollution free motorized vehicles. Dedicated pathway for non-motorized and mass transit is essential for them to share larger percentage of mobility demand in Dhaka. As the demand for mass transit grows, electric powered light rail system should be introduced to replace buses in major corridors of travel. Gasoline and CNG powered private cars should be discouraged through introduction of high import and congestion taxes for those vehicles. Government may also introduce a VKT (Vehicle Kilometer Travelled) tax to discourage driving. In order to promote electric and alternative (carbon free) fuel vehicles, government may consider tax benefits for production, sale, and purchase of these vehicles, especially when a zero emission vehicle replaces a fossil fuel based one.
5 HIGH LEVEL POLICY CONSIDERATIONS

Though Dhaka seems to be in a complete traffic chaos already, we may not have seen the worst of it. As the mobility demand on the scarce city road network continues to grow with the growth in population and economic activities, traffic is expected to become more chaotic. There is still time to plan and prepare for the future transportation demand and mobility before it gets completely out of control. That plan has to be comprehensive, coordinated, and sustainable. A forward looking mobility plan must take into account everything that have impact on and are impacted by transportation, making it comprehensive. It must identify the players and the processes for execution of the plan, both from capital development and long-term operation perspectives. Implementation and operation should be based on sustainability. It is not enough to simply provide transportation and mobility if it cannot be sustained.

Addressing mobility issues begins at the top of the government where the legislature must set the policy and a clear direction, first. Government of Bangladesh has taken the first step by creating the Dhaka Transport Co-ordination Authority (DTCA) through a legislative act, Dhaka Transport Coordination Authority Act, 2001 following a recommendation that came out of the World Bank funded Dhaka Urban Transport Project (DUTP). DTCA is created with the right vision, but it is dependent on many other government agencies for the implementation of its mission (DTCA 2014). In order for DTCA to be effective in its mission, it needs to have an independent governing body that can focus on DTCA activities. DTCA must also have the right and adequate staff resources for planning, promoting, funding, coordinating, and managing the execution of mobility related projects.

Because Bangladesh has a centralized government that sits in Dhaka where all government and most economic activities take place it is unconceivable how the pressure of population and mobility demand on Dhaka can be eased without creating strong urban centers around the country where people can participate and enjoy similar economic and social activities that Dhaka offers. Strong local and regional governments with jurisdiction to set, collect, and spend tax revenues, develop and execute regional land use and transportation plans, and create business and job opportunities can help redistribute urban population resulting in eased population pressure on Dhaka.

6 CONCLUSIONS

Dhaka’s traffic woe is the result of lack of proper land use and transportation planning and has been exacerbated by lack of traffic management and law enforcement. Though Dhaka’s traffic situation is desperately anarchic, some level of order can be restored with innovative traffic management approach. Fast action with a long-term plan that lays out gradual implementation is necessary. Whatever funds Bangladesh can muster for infrastructure development must go to high priority projects like Padma Bridge, power plants, and deep sea port leaving not much for expensive transportation projects for Dhaka. Also, unplanned organic growth of Dhaka didn’t leave much room for new transportation infrastructures. In addition to the current development plan for the new Mass Rapid Transit (MRT) lines, planners and policy makers can implement a few immediate traffic management approaches in order to bring some order in the disorderly traffic in Dhaka. Traffic problem in Dhaka is unique because of the unparallel mixed mode (motorized and non-motorized) mobility demand created by the most densely populated urban area in the world. A isn't any solution example for a problem of this proportion that Dhaka could readily adopt. Dhaka has to come up with its own solution. As suggested here, Dhaka’s solution approaches may include separation of modes within the given roadway space, one-way traffic movement on most narrow local streets, and a set of TDM measures. Effective separation of modes with adequate roadway width allocated to non-motorized and public transportation will likely generate the most traffic management benefits by introducing conflict free traffic, removing unauthorized roadside parking, enforcing bus stoppage at dedicated space, and introducing special lanes for non-motorized rickshaw and bicycle traffic. In addition, implementing one-way street network within the local neighborhoods will help traffic flow more smoothly in the very limited transportation space and narrow streets. Other TDM measures such as effective traveler information and innovative tax for the driving privilege may also incentivize less driving. Long-term policy strategies for management of population growth in Dhaka and empowering DTCA would help to alleviate traffic congestion in Dhaka. Mode separated thoroughfare along with one-way streets can be first implemented as a test case along one corridor. A test bed will help learn and refine the design and implementation. Dhaka must try a comparatively inexpensive and
sustainable mobility solution that proportionately emphasizes both motorized and non-motorized transportation according to their mode share. Expensive transportation solutions can come later as the economy grows with a sustainable mobility in place.

REFERENCES


Implementation of Thermoelectric effect to Road Facilities

<table>
<thead>
<tr>
<th>AUTHOR (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>LEE, Jaejun</td>
<td>Assistant Professor</td>
<td>Chonbuk National University</td>
<td>Republic of Korea</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CO-AUTHOR(S) (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>KIM, Daehoon</td>
<td>Graduated Student</td>
<td>Chonbuk National University</td>
<td>Republic of Korea</td>
</tr>
<tr>
<td>KIM, Seunghoon</td>
<td>Graduated Student</td>
<td>Chonbuk National University</td>
<td>Republic of Korea</td>
</tr>
<tr>
<td>KWON, Sooahn</td>
<td>Senior Research Fellow</td>
<td>KICT</td>
<td>Republic of Korea</td>
</tr>
<tr>
<td>HONG, JaeCheong</td>
<td>Research Fellow</td>
<td>KICT</td>
<td>Republic of Korea</td>
</tr>
<tr>
<td>KIM ByungJin</td>
<td>Road Management Division</td>
<td>MOLIT</td>
<td>Republic of Korea</td>
</tr>
</tbody>
</table>

E-MAIL (for correspondence)  
lee2012@jmu.ac.kr

KEYWORDS:  
Thermoelectric, temperature difference, Asphalt Pavement, Energy harvesting

ABSTRACT:  
Energy Harvesting stands alone as one of the most interesting techniques for approaching the global energy problem without depleting natural resources. Energy harvesting technologies from road infrastructure is new research areas that encompasses technologies that collect the wasted energy occurred in road space such as pavements, external space and store it for later use. Energy surrounds road space is available in many different forms, such as wind and solar energy or thermal and mechanical energy. In this study, we are trying to develop an energy harvesting system that can be installed in the barrier on the road. The system can capture energy from the temperature difference between the barrier surface and the barrier inner using thermoelectric modules. This paper presents a trial energy harvesting system on barriers for energy harvesting system on the road. The system is focus on the development of energy harvesting system for energy utilization. The system includes a 4cm, 4cm, 5cm, cooling pad which is to make more temperature difference. It is embedded in concrete specimen.
Implementation of Thermoelectric effect to Road Facilities

Dr. Jaejun Lee¹, Daehoon Kim¹, Seunghoon Kim¹
Dr. Sooahn Kwon², Jaecheong Hong², ByungJin Kim³

¹Chonbuk National University, Jeonju, Jeonbuk, Republic of Korea
²Korea Institute of Civil Engineering and Building Technology, Goyang, Gyeonggi-do Republic of Korea
³Ministry of Land, Infrastructure and Transport, Sejong, Republic of Korea

Email for correspondence: mail to: lee2012@jbnu.ac.kr

1. INTRODUCTION

Energy surrounds us and is available in many different forms, such as wind and solar energy or thermal and mechanical energy. One of these “naturally occurring” energy sources is the asphalt pavements that all day long receive lots of solar energy which gets dissipated as thermal energy at their inner structure. The resulting augmented temperatures with the traffic loads affect dramatically the surrounding environment and the service life of the pavements through raveling and rutting incidents. What makes the concept of harvesting energy from pavements enticing is that they offer an existed infrastructure that its dimensions are countless.[Symeon, 2012]

Mallick et al. (2009) emphasized that enhanced asphalt pavements that include an energy harvesting system, to reduce Urban Heating Island(UHI) effect, also clearly decrease rutting in asphalt pavements. In their study, they has conducted a large scale experiment to investigate the interaction among mechanisms like conduction, convention and radiation with the engineering parts of the whole system like the geometry of the pipes, the temperature of the inner water and rate of the flow fluid. Both wind speed and solar radiation were measured and solar radiation data were simulated with time. By using only one pipe and a particular range of fluid flow rate, temperatures of several points in slab were collected. In order to determine the temperature at different levels into the pavement, the experimental setup was modelled in finite element method.

In general, they found good correlation between the experimental data from the slab and theoretical data from the simulation model regarding the input temperature at the pavement system and the surrounding air. They concluded that the distribution of the temperature at the slab and the cooling of the surface pavement is the diameter of the pipe were affected, however, the flow rate of the fluid did not affect significantly the temperature of the surrounding space and the slab. They found that the larger the pipe diameter becomes, the steeper temperature variation occurs from pipe to pavement surface as the larger diameter results in lower level of water temperatures and a higher rate of lowering the pavement surface.[Mallick et al. 2009; Mallick et al. 2011a; Mallick et al. 2011b; Symeon, 2012]

Xu et al. (2012) researched hydronic heating used to prevent ice formation and snow accumulation on road surface pavements with the purpose of improving driver safety. They adopted an extended Darcy law and continuity equation to describe water flow cause by moisture and temperature gradients. The developed model was capable of providing good simulations of the evolution of temperature and surface conditions during snowmelt. Simulation comparisons indicate that including the effects of melted snow on thermal properties is important in the simulation of temperature and surface condition during snow melting.

Photovoltaics was extending into new energy harvesting market. Photovoltaic noise barriers(PVNB) along motorways and railways permit one of the most economic applications of grid-connected PV with the additional benefits of large scale power plants (typical installed power: more than 100 kWp) and no extra land consumption. Nordmann et al.[2000] had researched to reveal the large potential that could be interested for PV on noise barriers with the overall objective of raising the share of renewable energies for the EU’s electricity market. In contrast to many PV-potential studies published before, this proposal is focusing on PVNB only, as one of the cheapest ways to implement large scale grid-connected PV installations.

As Wu et al. (2011) underlined “thermal collection starts as long as the temperature of the location of the pipes reaches the balance temperature needed by specific heat transfer flow rate, wind speed, irradiation intensity and other conditions”. The cooled pavement surface can be improved its stiffness specifically in hot climatic conditions and may reduce or prevent permanent deformation, and hence extend the life of the pavement. [Stempilha, et al. 2012; Zhu, et al. 2011].

Traditional thermoelectric systems are comprised of a number of doped semiconductor elements arranged electronically in series and thermally in parallel as shown in FIGURE 1. If heat is flowing between the top and bottom of the thermoelectric device (forming a temperature gradient) a voltage will be produced and hence an electric current
Harvesting thermoelectric energy mainly relies on the Seebeck effect that utilizes a temperature difference between the two ends of the device for driving the diffusion of charge carriers.

2. THERMOELECTRIC TECHNOLOGY
Thermoelectricity (TE) is the conversion of heat into electricity (Seebeck effect), or of electricity into heat or refrigeration (Peltier effect). The use of the Seebeck effect could allow heat to be saved which would be otherwise lost. Although the conversion efficiency is very low, it has been enjoying renewed favour for several years, and novel research and development leads have been investigated, such as new materials and the structuring of matter at the nanoscale. This combination has led to active investigations worldwide, but without achieving the decisive breakthrough, which will give TE a prominent place among energy harvesting technologies. The most promising applications of TE, in the context of energy saving, concern thermal engine heat recovery (particularly in transport applications), and human body heat scavenging to power portable devices. TE for energy harvesting has several barriers to overcome: low conversion efficiency; toxicity; and low availability of chemical elements constituting part of the most interesting thermoelectric materials. In this context, the main challenges for nanotechnology are to demonstrate high efficiency improvement, and to display low cost implementation in thermoelectric materials.

Xi et al. (2007) investigated a solar-based driven thermoelectric technology and their applications were presented. Initially, a brief analysis of the environmental problems related to the use of conventional technologies and energy sources was appeared and the benefits offered by thermoelectric technologies and renewable energy systems were outlined. Figure 1 describes a structure of thermoelectric module.

![Figure 1: The basic structure of a thermoelectric module (TEM)](image)

2. EXPERIMENTAL SET UP

2.1 Experimental Design

The aim of this paper is to apply thermoelectric energy harvest technology on road facilities as seen in Figure 2. As seen in Figure 2, concrete barrier was always exposed on sunlight for daytime. The thermoelectrical module (TE) represent a technique that harvests energy from the different temperature based on Seebeck effect that utilizes a temperature difference between two ends of the devices for driving the diffusion of charge carriers. TE is possible to be used to capture “free” waste heat for electricity generation. [Wu et al. 2012a; Wu et al. 2012 b.]. Wu et al.(2012a) has studied the implementation of thermoelectric modules on the surface of the pavements and by conducting simulations.
they tried to optimize their design. The important observation behind the efficiency is a guaranteed high temperature difference between the upper and lower surface of the thermoelectrical (TE) module. The research team considers concrete barrier which was always received the solar energy for day time. So, our aim is to validate the energy harvest from barrier on highway as seen in FIGURE 2. As seen in FIGURE 2, the concrete barrier was always received the solar on the road. The research team considers the energy harvesting from surfaces that serve as solar collectors from surface of concrete barrier. Experiment was conducted in the laboratory using small specimen and halogen lamp to simulate concrete barrier and sunlight. Figure 3 shows the entire experimental set up. Figure 3(a) shows the data logger and data collect system. The output from TE was connected to data logger. Figure 3 (b) explains the halogen lamp 75W was used to simulate the solar energy to heat the surface of the TE module or cover plates.

![Data collect system](image1)

(a) Data collect system

![Simulated solar light](image2)

(b) Simulated solar light

![Thermoelectric module and cooling sink](image3)

(c) Thermoelectric module and cooling sink

![Concrete specimen with or W/O cooling sink](image4)

(d) Concrete specimen with or W/O cooling sink

**FIGURE 3 Experimental set up**

Figure 3 (c) shows a thermoelectric module is installed in the surface of concrete specimen and cooling sink to increase temperature difference between top and bottom of the thermoelectric module. The thickness of TE module involved here is only 0.50cm. Figure (3) (d) shows the TE case to mount on concrete surface. Based on thermal conductivity, aluminum material was adopted to build case for TE generator. Then the thermoelectric module with aluminum case was installed on the surface of concrete specimen, with or without the aluminum cover (as described in Figure 3(d)). Two temperature signals (temperatures of upper TE module surface and lower TE module) are monitored through Dewesoft 7.0 and Dewasoft’s DS-NET data acquisition to collect voltage data of the TE module output. The experiment duration was between about 5 hours. Our general idea is to install the thermoelectric (TE) module at the surface of barrier to utilize its high surface temperature.
3. TEST RESULTS

3.1 Case 1: using cooling sink

Figure 4 shows test set up using cooling sink or not to produce voltage on concrete specimen. The experiment duration was about 5 hours. As is shown in Figure 4(a), TE moduli were placed on cooling sink (Type B) or on concrete surface (with cooling sink, Type A). The cooling sink was embedded into concrete specimen due to keep same distance between lamp and surface of TE modulus. Figure 4 (b) plots the generated voltage verse time. From this figure, it can be seen that the produced voltage was significantly different as function of conditions, Type A and Type B. The slope first increase, reaches its highest value, and then slightly decreased from both Type A and Type B. After highest value, the generated output was decreased because of decreasing temperature variation between top and bottom of TE modulus. The lamp energy can be transmitted the TE modulus. There was significantly different between Type A and Type B as shown in Figure 4 (b). Type B with cooling sink was generated more voltage than that of Type A. Compared Type A with Type B of high value, Type B was 1.5 times higher than that of Type A. It indicated that cooling sink generates more temperature difference between surface and bottom of TE modulus. As increased temperature gap, the produced energy was higher.

![No cooling sink](image1.png) ![Cooling sink](image2.png)

Type A  Type B

(a) Temperature  (b) Voltage

FIGURE 4 Monitored Temperature and Thermal output

3.2 Case 2: Development of TE case

The design and application of thermoelectric generator needs to be optimized for the maximal efficiency. The geometry design needs to ensure there is sufficient amount of heat flow that can be collected and converted, so that a relatively large thermal gap can be maintained. In this study, the research team designed the thermoelectric device case to improve efficiency as seen in Figure 5. Figure 5 (a) and (b) show the TE case’s photos and Figure 5 (c) and (d) describes detailed TE cases. The case is required to install on concrete barrier to prevent damage of TE modulus. As described in Figure 5 (c) and (d), the geometry of case was 5 cm height, 5 cm wide, and 5 cm length that was made with aluminum plate. Figure 5 (b) shows the voltage of the harvester from TE modulus. As plotted at Figure 5 (b), the Case A is represented the higher output voltage than that of Case B. The peak value of Case A is two times higher than that of Case B. As seen in Figure 4 (b), the same trend was observed at Figure 5 (b). The slope was dramatically increased to higher peak as time passes by. Case A shows two times higher than that of Case B as described in Figure 5. It indicates that the cover made by aluminum should be directly contacted on surface of TE modulus. If there was a space between the cover and the TE modulus, the efficiency of the TE is decreased because of thermal conductivity.

![Case A](image3.png) ![Case B](image4.png)

(a) Housing Set up  (b) Voltage
4. CONCLUSION AND FUTURE RESEARCH WORKS

Harvesting energy in situ is near ideal for road facilities applications such as concrete barriers. This paper presented the implementation of a novel thermoelectric energy harvesting technology that harvest energy using the temperature difference between the surface and bottom of the thermoelectric module on road facilities. The conceptual framework for thermoelectric power generation from the thermal difference of road facilities is presented in this paper. The test set up performance was evaluated in simulated laboratory conditions. This study validated the feasibility of the TE concept as potential energy source for road facilities. The final destination of this study is drive TE energy to improve road safety and to improve pavement service life in the future. Also, the optimum TE energy harvesting case will be developed in the future.

ACKNOWLEDGEMENT

This research was supported by Basic Science Research Program through the National Research Foundation of Korea(NRF) funded by the Ministry of Science, ICT & Future Planning(2014R1A1A1004577)

REFERENCES


Symeonis, Andreoiopolou. (2012) A review on Energy harvesting from roads, MSc Environmental Engineering & Sustainable Infrastructure


Developing a Trafficability Index of Vehicles during Winter

Younshik CHUNG  
Research Fellow  
The Korea Transport Institute  
Korea(ROK)

E-mail  
tpgist@koti.re.kr

Keywords:  
Friction, Trafficability, Roadway Surface Condition, Trafficability Index, Weather Information, Information Convergence

Abstract:  
Information about trafficability, or the condition of a roadway section with regard to its being traveled over by vehicles, is one of the most critical factors for roadway operation in winter. Specifically, when traveling on snowy or icy surfaces, the traction varies per vehicle types, tire types, geometric characteristics of the roadway, and conditions of the roadway surfaces. Thus, traffic information regarding trafficability with respect to vehicle types, geometric characteristics of roadway sections, and roadway surface conditions can provide a foundation to make a decision whether to use the associated roadway sections for roadway operators as well as users. Based on the preceding premise, the objective of this study is to present a methodology for developing a trafficability index with respect to vehicle types, geometric characteristics of roadway sections, and roadway surface conditions. Two datasets were combined to accomplish the objective of the study: (1) traction data from previous studies, and (2) road geometry data obtained from suburban area in Seoul metropolitan area, Korea.
Developing a Trafficability Index of Vehicles during Winter

Younshik Chung
The Korea Transport Institute, 315 Goyangdae-ro, Ilsanseo-gu
Goyang, Gyeonggi 411-701, Korea
Tel: + 82-31-910-3243; Fax: + 82-31-910-3228; E-mail: tpgist@koti.re.kr

1 INTRODUCTION

One of the most critical problems for roadway operators in the winter may be the snow removal work. Apart from the snow removal issue, roadway operators and users are commonly faced with the decision making if a specific section such an uphill section will be trafficable when the snow was forecasted. The problem regarding the decrease of trafficability mainly erupts in uphill sections rather than on plains. In other words, a snowfall on an uphill section of a roadway reduces the friction between the vehicle and the roadway thus making the uphill section impossible to climb or safely descend. In particular, few drivers would expect that they can climb the uphill section by using the power of vehicle’s acceleration. However, when such trials lead to fail in roadways with a small number of lanes, there could be traffic congestion or vehicle’s isolation due to hampering the entry of other vehicles and snow removal vehicles. This problem is likely due to the lack of information with respect to the trafficability of roadway sections.

In general, even if the surface conditions and the geometric characteristics of the roadway are the same, the trafficability of vehicles varies with the vehicle's performance. In other words, in spite of the same roadway conditions and the geometric characteristics, certain vehicles cannot travel the roadway section, but others can. Thus, if information regarding weather conditions or roadway conditions and the geometric characteristics of the roadway is fused, the trafficable index for the winter roadway can be built for the purpose of the roadway operation and vehicle control. Although such an index-based information is expected to be used as highly useful, none of the previous studies has been found based on the literature reviewed by the author. With this background, the objective of this study is to develop a roadway trafficability index based on information regarding the roadway geometry, vehicle characteristics, and roadway surface conditions. To accomplish this objective, this study used the existing research results on the roadway friction with respect to vehicle types.

2 LITERATURE REVIEW

2.1 Traction performance

Generally, the possibility of whether a particular vehicle can or cannot pass through the particular roadway section can be determined by three factors: roadway grade, vehicle's driving force, and road surface conditions regarding the weather conditions.

![Figure 1 Forces for a vehicle at its maximum climbing ability](image)

Physically, the force when a vehicle is on a roadway is the grade of the roadway sections, which consist of the force acting in the direction towards the ascent of the section (vehicle driving force), and the resistance force sliding towards the downhill section as shown in Figure 1 and these two forces are represented by the following formula (Raad and Lu, 2000; Raad and Lu, 1998):
- Driving force: $\mu mg \cos \theta$
- Resistance force: $mg \sin \theta$

where $m$ represents the mass of the vehicle, $g$ is the ground acceleration, $\mu$ is the rolling resistance coefficient, $\theta$ is the slope of roadway section.

Thus, the maximum climbing degree of a vehicle is the point when the driving force and the resistance force are equal. This relationship can be expressed as follows:

$$\mu mg \cos \theta_{\text{max}} = mg \sin \theta_{\text{max}}$$

where $\theta_{\text{max}}$ represents the maximum climbable degree. In this equation, the vehicle mass varies with the type of vehicles and the coefficient of the rolling resistance can be measured through experiments. Furthermore, the variable $g$ is generally applied to the acceleration of gravity and $mg \cos \theta$ is described as the maximum static friction force. However, $g$ is applied to the acceleration of the vehicle, since it is describing the driving state of the vehicle.

![Figure 2 Forces for a vehicle on a level surface](image)

Based on the foregoing description, the maximum trafficability of the vehicle can be derived for a particular type of vehicle on a particular roadway section condition. This situation is shown in Figure 2. That is, the maximum traction or maximum $G$ ($G_{\text{max}}$), defined as the average friction coefficient between the tires and the ground surface, can be derived when $\theta = 0$ (Raad and Lu, 2000; Raad and Lu, 1998). Thus, above equation (1) can be rewritten as:

$$mgG_{\text{max}} = mg \mu$$

or

$$G_{\text{max}} = \mu$$

In addition, by using equation (1) and (3), the following equation (4) can also be derived. As a result, once the $G_{\text{max}}$ values for the individual vehicles and the surface conditions for each roadway section can be found, the maximum trafficable degree for individual vehicles with respect to various roadway surface conditions can be identified. Therefore, equation (4) shows that increasing the friction coefficient results in greater the trafficability degree (or climbing angle).

$$G_{\text{max}} = \tan(\theta_{\text{max}})$$

3 PREVIOUS STUDIES

Research related to the friction between the road surface and vehicle has been mostly focused on the type of tires based on the characteristics of friction. In Europe and North America, such studies were conducted from the 1960s and early 1970s (Fromm and Corkill, 1971; Greek, 1975; Rosenthal et al., 1969; Smith and Schonfeld, 1970; Smith et al., 1971), and most of them were focused on measuring the stopping distance. The results from these studies showed that studded tires reduced the stopping distance on icy roads but increased the stopping distance on dry or wet roads.

From the 1980s, along with advancements in vehicle technology and performance, studies associated with the friction between vehicle and road surface included new factors. That is, driving support systems such as four-wheel-drive vehicles and anti-lock braking system (ABS) were newly introduced in experiments. Hayhoe and Kopac (1982) conducted a study on the braking friction and traction force of general tires, snow tires, studded tires, four-wheel-drive vehicles, four-wheel and ABS-equipped vehicles and rear-wheel and ABS-equipped vehicles. According to the results of their experiments, studded tires were found to have good braking performance on icy roads and both snow tires and studded tires were shown to
have the same performance on snowy roads, but on wet roads all equipments had the same performance. On the other hand, in the case with traction force, four-wheel drive systems had the most excellent performance on icy and snowy roads, followed by studded tires. Other equipments were shown to have the same performance. In addition, four-wheel drive vehicles were shown to have the most remarkable performance on wet roads and other equipments had the same performance.

In the 1990s, more advanced tire technology was developed. As a result, the number one tire manufacturing company, Bridgestone, launched a new type of tire called Blizzak. The researchers at the University of Alaska in the United States performed an experiment related to vehicle traction force both on snowy and icy roads regarding three types of tires (Lu et al., 1994). Table 1 represents the results of their experiments.

Table 1 Example of average traction study results

<table>
<thead>
<tr>
<th>Type of tires</th>
<th>Blizzak</th>
<th>Studded</th>
<th>All-season</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traction test</td>
<td>Snow</td>
<td>Ice</td>
<td>Snow</td>
</tr>
<tr>
<td>Stopping distance</td>
<td>19.5</td>
<td>36.5</td>
<td>32.3</td>
</tr>
<tr>
<td>(40.2km/h)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Starting traction</td>
<td>9.6</td>
<td>14.4</td>
<td>9.1</td>
</tr>
<tr>
<td>(time in sec to</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>reach 42.2 km/h)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum cornering</td>
<td>27.7</td>
<td>22.8</td>
<td>25.6</td>
</tr>
<tr>
<td>speed (15.2m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>radius in km/h)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum starting</td>
<td>16</td>
<td>11</td>
<td>16</td>
</tr>
<tr>
<td>grade (%)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In the late 1990s, Raad and Lu (1998) evaluated the winter traction performance of transit and paratransit vehicles including a 9-passenger van, 1 30-passenger bus, and a 35-passenger bus. In this study, stopping distance, starting traction, maximum hill-climbing ability, and degree of vehicle movement direction change occurring with sudden stops were tested with respect to both snow and icy road surfaces. Recently, vehicle traction characteristics were investigated on the more diverse road surfaces (Sokolovskij, 2007). Specifically, this study was tested by classifying the road surfaces into 12 cases rather than 2 cases (i.e., snow and icy surfaces).

As we have seen so far, the research related to the friction between the road and the vehicle or the traction has been mostly conducted to estimate the friction coefficient on simple changes in the external environment. In the field of transportation engineering, the friction coefficient is mainly used for the purpose of the accident prevention during winter due to reducing the friction between the road surface and the vehicle (Scheibe, 2002; Sokolovskij, 2007). In other words, limited studies have been accomplished to establish the operational criteria for the road networks and vehicles with respect to the geometric characteristics of the roadways, vehicle types, and weather conditions.

With this background, this study develops a basic information to make a decision whether or not a specific type of vehicles will pass a specific roadway section under the consideration of various external conditions such as the geometric condition, vehicle performance, and weather conditions during winter. The friction coefficients estimated by previous studies were used to accomplish this purpose.

4 TRAFFICABILITY INDEX

4.1 Average maximum friction coefficient of vehicles

The friction coefficient for various types of vehicles can be obtained by experiments. However, this study used the values from the maximum average friction coefficient ($G_{max}$) suggested by Raad and Lu (2000). Specifically, the type of vehicles for the trafficability index was simplified to passenger car, vans, bus, and light truck. However, the values of other vehicles not shown in Table 2 can be estimated by using the interpolation method based on the vehicle weight.

Table 2 Examples of $G_{max}$ by vehicle types on non-battered snow

<table>
<thead>
<tr>
<th>Type of vehicles</th>
<th>Vehicle weight</th>
<th>$G_{max}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passenger car</td>
<td>1.5t</td>
<td>0.14</td>
</tr>
<tr>
<td>Van</td>
<td>4.6t</td>
<td>0.15</td>
</tr>
</tbody>
</table>
4.2 Average maximum friction coefficient of vehicles

Traction implies to a machine’s ability to continue moving forward without the wheels slipping (Burch, 2004), and it refers the maximum friction that can be produced between the road surface and vehicle without slipping. In addition, the coefficient of traction is defined as the usable force for traction divided by the weight on the vehicle. Thus, the coefficient of traction (φ) is a critical variable that can determine whether or not a vehicle can climb an uphill roadway section without slipping. The traction coefficient is also affected by the type of pavements and weather conditions. In this study, the pavement type of the road was assumed as asphalt and the traction coefficient for weather conditions used the results from the study by Sokolovskij (2007). Table 3 shows the traction coefficient (φ) of vehicles with respect to the road surface conditions by Sokolovskij.

Table 3 Traction coefficient of universal tire by road surface conditions

<table>
<thead>
<tr>
<th>Road surface</th>
<th>More detailed description of the surface condition</th>
<th>Traction coefficient φ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Battered snow</td>
<td>Snow, battered by automobiles, which does not make the pounded layer of snow and ice</td>
<td>0.24~0.37</td>
</tr>
<tr>
<td>Non-battered snow</td>
<td>Snow, which has just fallen on the asphalt and which is not battered by the wheels of automobiles - the first driving</td>
<td>0.15~0.42</td>
</tr>
<tr>
<td>Snow and ice, covered with the snow, which has just fallen</td>
<td>Battered snow and ice, covered with the layer of snow (thickness – up to 10 cm), which has just fallen and is not battered</td>
<td>0.18~0.45</td>
</tr>
<tr>
<td>Snow and ice, mixed with sand and slush</td>
<td>Battered snow and ice, mixed with sand and slush, the particles of which make 3~6 mm in diameter</td>
<td>Depending upon the quantity of slush (little–much) 0.15~0.45</td>
</tr>
<tr>
<td>Snow and ice</td>
<td>Entire layer of snow, battered to the extent of the icy surface</td>
<td>0.12~0.39</td>
</tr>
<tr>
<td>Snow and ice before crossroads</td>
<td>Snow, which at first was melted by the motors of the standing automobiles and then frozen up to the smooth surface</td>
<td>0.09~0.22</td>
</tr>
<tr>
<td>Dry asphalt in winter conditions</td>
<td>Dry asphalt (not covered by anything) in winter conditions</td>
<td>0.59~0.72</td>
</tr>
<tr>
<td>Asphalt, covered with hoar-frost</td>
<td>White cover on the asphalt, which is observed by the driver and easily recognized as hoar-frost</td>
<td>0.48~0.58</td>
</tr>
<tr>
<td>Smooth ice</td>
<td>Thick layer of frozen water, non-infringed with prickles and chains</td>
<td>0.054~0.19</td>
</tr>
<tr>
<td>Ice and tires with chains</td>
<td>Thick non-infringed layer of frozen water, infringed with the wheels, equipped with steel chains</td>
<td>0.12~0.18</td>
</tr>
<tr>
<td>Black ice</td>
<td>Thick ice layer, looking as a wet, black stretch of the road, which seems fit for traffic, and is not easily noticed by the driver</td>
<td>0.12~0.26</td>
</tr>
</tbody>
</table>

4.3 Trafficable slope

The values proposed in Table 2 refer to the average maximum friction coefficients (Gmax) of vehicles on non-battered snow. The Gmax values for passenger vehicles appeared to be 0.14. On the other hand, the values presented in Table 3 displays the traction coefficients (φ) of universal tires with respect to the road surface conditions. As described by Sokolovskij (2007), the range of the traction coefficient for the case of...
non-battered snow in Table 3 is comparatively wide (i.e., 0.15~0.42) according to the experimental conditions, tire conditions, weather conditions, and other variables. Although the spatio-temporally experimental environment to obtain the result of Table 2 and Table 3 was different, the $G_{\text{max}}$ value is very similar to the lowest value of $\phi$. As described above, therefore, the average maximum friction coefficient can be referred as the traction coefficient.

However, the use of conservative values in the rage of the $\phi$ value is recommended in the perspective of safe traffic operation and control strategies. Thus, this study applies the lowest value in the rage values. Based on this idea, the average maximum friction coefficient $G_{\text{max}(i,j)}$ for the road surface condition $i$ and for the vehicle type $j$ can be calculated by using the $G_{\text{max}}$ value in Table 2 and the $\phi$ value in Table 3. Table 4 represents the calculated $G_{\text{max}(i,j)}$, and

Table 4 shows the degree of maximum slope with respect to vehicle types and road surface conditions, which is obtained by using the unit conversion to degree (°) and equation (4).

<table>
<thead>
<tr>
<th>Road surface condition $i$</th>
<th>$G_{\text{max}(i,j)}$ for vehicle type $j$</th>
<th>Passenger car</th>
<th>Van</th>
<th>Bus</th>
<th>Light truck</th>
</tr>
</thead>
<tbody>
<tr>
<td>Battered snow</td>
<td></td>
<td>0.24</td>
<td>0.257</td>
<td>0.274</td>
<td>0.24</td>
</tr>
<tr>
<td>Non-battered snow</td>
<td></td>
<td>0.15</td>
<td>0.161</td>
<td>0.171</td>
<td>0.15</td>
</tr>
<tr>
<td>Snow and ice, covered with the snow, which has just fallen</td>
<td></td>
<td>0.18</td>
<td>0.193</td>
<td>0.206</td>
<td>0.18</td>
</tr>
<tr>
<td>Snow and ice, mixed with sand and slush</td>
<td></td>
<td>0.15</td>
<td>0.161</td>
<td>0.171</td>
<td>0.15</td>
</tr>
<tr>
<td>Snow and ice</td>
<td></td>
<td>0.12</td>
<td>0.129</td>
<td>0.137</td>
<td>0.12</td>
</tr>
<tr>
<td>Snow and ice before crossroads</td>
<td></td>
<td>0.09</td>
<td>0.096</td>
<td>0.103</td>
<td>0.09</td>
</tr>
<tr>
<td>Dry asphalt in winter conditions</td>
<td></td>
<td>0.59</td>
<td>0.632</td>
<td>0.674</td>
<td>0.59</td>
</tr>
<tr>
<td>Asphalt, covered with hoar-frost</td>
<td></td>
<td>0.48</td>
<td>0.514</td>
<td>0.549</td>
<td>0.48</td>
</tr>
<tr>
<td>Smooth ice</td>
<td></td>
<td>0.054</td>
<td>0.058</td>
<td>0.062</td>
<td>0.054</td>
</tr>
<tr>
<td>Ice and tires with chains</td>
<td></td>
<td>0.12</td>
<td>0.129</td>
<td>0.137</td>
<td>0.12</td>
</tr>
<tr>
<td>Black ice</td>
<td></td>
<td>0.12</td>
<td>0.129</td>
<td>0.137</td>
<td>0.12</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Road surface condition $i$</th>
<th>Maximum slope for vehicle type $j$ in degree</th>
<th>Passenger car</th>
<th>Van</th>
<th>Bus</th>
<th>Light truck</th>
</tr>
</thead>
<tbody>
<tr>
<td>Battered snow</td>
<td>13.50</td>
<td>14.42</td>
<td>15.34</td>
<td>13.50</td>
<td></td>
</tr>
<tr>
<td>Non-battered snow</td>
<td>8.53</td>
<td>9.13</td>
<td>9.73</td>
<td>8.53</td>
<td></td>
</tr>
<tr>
<td>Snow and ice, covered with the snow, which has just fallen</td>
<td>10.20</td>
<td>10.92</td>
<td>11.62</td>
<td>10.20</td>
<td></td>
</tr>
<tr>
<td>Snow and ice, mixed with sand and slush</td>
<td>8.53</td>
<td>9.13</td>
<td>9.73</td>
<td>8.53</td>
<td></td>
</tr>
<tr>
<td>Snow and ice</td>
<td>6.84</td>
<td>7.33</td>
<td>7.81</td>
<td>6.84</td>
<td></td>
</tr>
<tr>
<td>Snow and ice before crossroads</td>
<td>5.14</td>
<td>5.51</td>
<td>5.87</td>
<td>5.14</td>
<td></td>
</tr>
<tr>
<td>Dry asphalt in winter conditions</td>
<td>30.54</td>
<td>32.30</td>
<td>33.99</td>
<td>30.54</td>
<td></td>
</tr>
<tr>
<td>Asphalt, covered with hoar-frost</td>
<td>25.64</td>
<td>27.22</td>
<td>28.75</td>
<td>25.64</td>
<td></td>
</tr>
<tr>
<td>Smooth ice</td>
<td>3.09</td>
<td>3.31</td>
<td>3.53</td>
<td>3.09</td>
<td></td>
</tr>
<tr>
<td>Ice and tires with chains</td>
<td>6.84</td>
<td>7.33</td>
<td>7.81</td>
<td>6.84</td>
<td></td>
</tr>
<tr>
<td>Black ice</td>
<td>6.84</td>
<td>7.33</td>
<td>7.81</td>
<td>6.84</td>
<td></td>
</tr>
</tbody>
</table>
4.4 Slope of roadway segment

The slope of the roadway segment in the suburban area of Seoul metropolitan area, Korea has been estimated by using a geographic information systems (GIS) modeling with the ArcMap software by ESRI (Environmental Systems Research Institute). To estimate the slope value, two data files were used: CAD (computer-aided design) drawing file for the principal facilities of cities and digital roadway map file. In general, the digital roadway map does not include the elevation data of the roadway, but the CAD drawing file has. Thus, the first step was to capture the principal elevation spots of the city area from the CAD file scaled in 1:1000. The 3D Analyst tools of the ArcMap enables to create the triangulated irregular network (TIN) from the captured elevation spots. Figure 3 shows the created TIN based on the elevation spots of a city.

![Image](image1)

(a) Principal elevation spots of a city area

![Image](image2)

(a) TIN created by principal elevation spots

Figure 3 CAD drawing data and TIN based on principal elevation spots of a city

The TIN data is applied to construct topographical features on the slope by using the 3D Analyst tools and the Spatial Analyst Tools. Figure 4 (a) depicts the shape file including slope information. The last step is clipping the roadway map in Figure 4 (b) from the topographical features on the slope in Figure 4 (a). Finally, Figure 5 represents the roadway slope map with small segments clipped by the topographical features.

![Image](image3)

(a) Converted topographical features on the slope

![Image](image4)

(b) Roadway map

Figure 4 Topographical features on the slope and roadway network of a city area
4.5 Trafficability index

By comparing the trafficable slope for the vehicle types and the road surface conditions with the slope of the roadway segment, the trafficability of each vehicle can also be determined. If such information is delivered to drivers as traffic information in real time or in advance based on weather forecasts, drivers can make a decision on the basis of a binary case (i.e., possible or impossible). Therefore, the development of a ‘trafficability index’, which includes the concept of the probability for decision making, is required.

The Trafficability Index \( TI_{ijk} \) of the road surface conditions \( i \) of vehicles \( j \) on road segments \( k \) can be obtained based on the \( G_{\text{max}} \) as follows:

\[
TI_{ijk} = 1 - \frac{\tan(\theta_i)}{G_{\text{max}(i,j)}}
\]  

(5)

where \( G_{\text{max}(i,j)} \) represents the average maximum friction coefficient of vehicle type \( j \) on road surface conditions \( i \) and \( \theta_i \) represents the slope of road segments \( k \) in degree (°). Thus, in the case that the value of the trafficability index of vehicle type \( j \) on the road surface conditions \( i \) is \( TI_{ijk} \leq 0 \), it implies that the travel of the associated vehicle on the road segment \( k \) is impossible, and in the case with \( 0 < TI_{ijk} < 1 \), the travel is probabilistically possible, and finally in the case with \( TI_{ijk} = 1 \), the travel is possible. In other words, as the \( TI_{ijk} \) value is closer to 1, the vehicle’s travel is more possible and in the case with \( TI_{ijk} \) being smaller than 0, the possibility of trafficability is zero. Furthermore, \( TI_{ijk} \) being 1 means that \( \theta_i \) is 0, meaning that there is no inclined angle, so any travel is expected to be possible in the current condition. Finally, in the case that the value of \( TI_{ijk} \) is between 0 and 1, the Road Administration or the road management officials can determine whether or not the vehicle can pass.

5 CONCLUSIONS

This paper presented a procedure to develop a trafficability index as a countermeasure of the safe driving and safe traffic operation in winter. Although the developed index used the results from the previous studies, the presented parameter values can be constructed through the field test and the presented procedures. The developed trafficability index will be useful to determine whether or not the Road Administration and the road management officials close the associated roadway section when snowfall event occurs. Furthermore, if such an index is delivered to drivers as traffic information, they will be able to detour their initially planned route or prepare special equipment such as chains and snow tires in advance. As a result, the traffic accidents and congestion due to roadway slipperiness will be reduced.

The trafficability index during the winter season may be constructed in real time on the basis of the information regarding current road surface and weather conditions and the vehicle types. However, since
weather information is a kind of predictable one, the trafficability index can also be predictable based on the predicted weather information. Thus, the trafficability index can be developed on the nationwide road networks, and it is expected to be used by drivers who are planning winter vacations. Furthermore, for the case of the logistics industries, their transportation damage due to heavy snowfall is expected to be minimized by evading snowfall-expected sections where travel is impossible and by modifying their travel schedule. Finally, this study focused on the trafficability in uphill sections, but the study related to the diagnosis on the issues associated with the loss in braking ability in downhill sections is required as future research. Also, an in-depth study of the trafficability that may vary with the braking ability and performance of a vehicle is required and this detailed research is expected to be conducted focusing on mid-to-large-size trucks which degrade the uphill trafficability.

ACKNOWLEDGEMENT

This work was supported by the National Research Foundation of Korea (NRF) grant funded by the Korea Government (MSIP) (NRF-153-3100-3133-302-350)

REFERENCE


The use of fiber concrete for bridge construction

TRACK
Pavements & Materials

AUTHOR
N. RETNO SETIATI
Position
Researchers
Organization
Bridge and structure experimental station, Institute of road engineering Research and Development Agency Ministry of Public Works
Country
Indonesia

CO-AUTHOR(S)

e-mail
retno.setiati@pusjatan.pu.go.id
retnosetiati@yahoo.com

E-MAIL
(for correspondence)

KEYWORDS:
Fiber, concrete, plate, composition, ductility

ABSTRACT:
The use of concrete with the addition of fibers has been widely used in planning and construction in several countries. Concrete is concrete with fiber composite material consisting of ordinary concrete and other materials in the form of fibers. Fibers in concrete are useful to prevent the cracks so to make concrete is more ductile than ordinary concrete. This study aims to determine the effect of adding synthetic fibers to mechanical properties of concrete. In the study made by the size of the plate specimen 1150 x 1150 x 200 mm. The composition of the fiber used varies between 4 kg/m² and 5 kg/m². Based on the test results showed that the effect of the fiber in the concrete composition can inhibit the spread of the larger cracks, has high ductility, and can increase the bending capacity. Significantly, the concrete cylinder specimen with the composition of the fiber can lower towing capacity divided by 22% compared to cylindrical concrete without fiber. However, the addition of fiber to the plate can increase the load capacity and ductility compared to plates without fiber.
The use of fiber concrete for bridge construction

N. Retno Setiati

1Bridge and structure experimental station, Institute of road engineering
Research and Development Agency
Ministry of Public Works
Email for correspondence: retno.setiati@pusjatan.pu.go.id

1 INTRODUCTION

In the development of modern concrete construction, concrete required to be of high quality construction materials at the same high performance. For example, in the fresh concrete, easily prosecuted in foundry workable, low heat of hydration, relatively low shrinkage during drying, have high levels of connective initial time (acceleration) or delay (retardation) is good, and easily pumped to a higher place, a few demands that must be met quality and high-performance concrete. While the concrete has hardened, high strength concrete and high performance are required to have high compressive strength, good tensile strength, high early compressive strength, ductile behavior, airtight and water, resistant to abrasion and corrosion sulfate, penetration low chloride, low expansion, shrinkage, and durable. To improve the performance of concrete, there are several ways you can do, such as by the addition of fiber.

Fiber concrete is the concrete mixture composition coupled with fiber. In general, the fiber used in the concrete mix a measuring rod (5-500) μm with a length of about (25-60) mm. Fiber materials can be kind asbesitos, polypropylene, pieces of steel wire, or plant fibers (jute, coir, bamboo, palm fiber) (Trimulyono, 2004). Fibers in concrete prevents cracks that makes concrete more ductile than ordinary concrete. The use of fiber concrete aims to increase the tensile strength of concrete to resist tensile force caused by the influence of load, climate, temperature and weather changes. The addition of the fiber itself can reduce cracks that may arise due to changes in the weather.

In the kind of fiber concrete, concrete fiber types can be differentiated into two types, namely natural fiber concrete and synthetic fibers. Natural fibers are generally made of a variety of plants. Because of the nature generally easy to absorb and release water, natural fibers easily weathered so is not recommended to use the high-grade concrete or for specific use. Synthetic fibers are generally made of polymer compounds, has a high resistance to weather changes, has a melting point, tensile strength, and high flexural strength. These fibers are used for high-grade concrete and which will be used exclusively.

In the physical properties of concrete, the addition of fiber causes changes to the properties of the concrete. Compared with the same quality concrete without fibers, the concrete with fibers make it more rigid so far slump value and make a quicker initial bind. While in its mechanical properties, the addition of fiber to the optimum limit generally may increase the tensile strength and flexural strength, but lower compressive strength.

The addition of fibers into the concrete mix is intended to overcome the unfavorable properties of concrete. The basic idea is to provide additional fiber reinforcement in concrete fibers evenly distributed at random to prevent cracking due to loading (Sudarmoko, 1990).

Based on the studies that have been conducted showed that the addition of fibers into the mortar will decrease workability rapidly along with the addition of fiber concentration and fiber aspect ratio. To obtain optimal results, there are two things to note: (1) Fiber aspect ratio, i.e. the ratio between fiber length (l) and fiber diameter (d), and (2) Fiber volume fraction (Vf), i.e. the percentage of fiber volume added to each unit volume of concrete (Suhendro, 1990).

From the research that has been done can be concluded that by adding fiber into the concrete mix in addition to improved ability to resist bending, as well as the ductility (ability to absorb energy) also increased (Suhendro, 1990). In addition, by adding glass fiber into the concrete mix will enhance the tensile strength of concrete (Sudarmoko, 1991).

In other studies that have been done by Swammy et al, 1979 conclude that the presence of fiber in the concrete will increase stiffness and reduce deflection happened. The addition of fiber can also increase the plasticity of concrete, so that the structure will be protected from the sudden collapse due to over loading.

2 METHODOLOGY

This study aimed to determine the effect of fiber on the mechanical properties of concrete. In the study made by the plate specimen size 1150 x 1150 x 200 mm with number 9 pieces. The composition of the fiber used varies between 4 kg/m³ and 5 kg/m³. Split tensile strength of concrete is done by making a concrete cylinder size of 150 mm x 300 mm with number 9 cylinder. Split tensile strength testing of concrete for all test objects is done by following the procedures in accordance with ISO standard ISO 03-2491-2002 (Figure 1). Split tensile strength test is an alternative to the direct tensile strength.
Plate flexural testing refers to ASTM: Designation C 1550 – 04. Measurement of strain in the plate were performed using an Vishay (with strain-gauge-type sensor foil) and a data taker, while the deflection measurement is done by using a deflector meters and displacement gauge. Tests carried out after all the equipment attached. Steps and methods of testing the specimen plate is almost the same as the steps and methods of testing the test object beam. Giving force was also performed to test specimens were destroyed by loading intervals per 10,000 N. But unlike the beam specimen, the plate specimen, specimen in the fulcrum on all four sides, then burden placed on the middle of the field plate (Figure 2).

Characteristics of concrete compressive strength is 33 MPa. Fibers are added to the concrete composition is fiberglass with specifications can be seen in Table 1.

<table>
<thead>
<tr>
<th>No.</th>
<th>Characteristics</th>
<th>Properties of materials</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Base resin</td>
<td>Modified Olefin</td>
</tr>
<tr>
<td>2.</td>
<td>Length</td>
<td>60 mm</td>
</tr>
<tr>
<td>3.</td>
<td>Tensile strength</td>
<td>640 MPa</td>
</tr>
<tr>
<td>4.</td>
<td>Surface texture</td>
<td>Continuously embossed</td>
</tr>
<tr>
<td>5.</td>
<td>The amount of fiber per kg</td>
<td>33,000</td>
</tr>
<tr>
<td>6.</td>
<td>Bulk density</td>
<td>0,90 – 0,92</td>
</tr>
<tr>
<td>7.</td>
<td>Young Modulus</td>
<td>10 GPa</td>
</tr>
<tr>
<td>8.</td>
<td>Melting point</td>
<td>(150 – 165) °C</td>
</tr>
<tr>
<td>9.</td>
<td>Ignition point</td>
<td>Greater than 450 °C</td>
</tr>
</tbody>
</table>
Test specimens were made under normal conditions (without the addition of fibers) and concrete with the addition of fiber. Mechanical testing performed on the concrete after 28 days. Identification of the test object can be seen by Table 2.

Table 2. Identifications of specimen

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen identification</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>C1-0</td>
<td>The first cylinder without fibers</td>
</tr>
<tr>
<td>2</td>
<td>C2-0</td>
<td>The second cylinder without fibers</td>
</tr>
<tr>
<td>3</td>
<td>C3-0</td>
<td>The third cylinder without fibers</td>
</tr>
<tr>
<td>4</td>
<td>C1-4</td>
<td>The first cylinder with fibers 4 kg/m^3</td>
</tr>
<tr>
<td>5</td>
<td>C2-4</td>
<td>The second cylinder with fibers 4 kg/m^3</td>
</tr>
<tr>
<td>6</td>
<td>C3-4</td>
<td>The third cylinder with fibers 4 kg/m^3</td>
</tr>
<tr>
<td>7</td>
<td>C1-5</td>
<td>The first cylinder with fibers 5 kg/m^3</td>
</tr>
<tr>
<td>8</td>
<td>C2-5</td>
<td>The second cylinder with fibers 5 kg/m^3</td>
</tr>
<tr>
<td>9</td>
<td>C3-5</td>
<td>The third cylinder with fibers 5 kg/m^3</td>
</tr>
<tr>
<td>10</td>
<td>S1-0</td>
<td>The first plate without fibers</td>
</tr>
<tr>
<td>11</td>
<td>S2-0</td>
<td>The second plate without fibers</td>
</tr>
<tr>
<td>12</td>
<td>S3-0</td>
<td>The third plate without fibers</td>
</tr>
<tr>
<td>13</td>
<td>S1-4</td>
<td>The first plate with fibers 4 kg/m^3</td>
</tr>
<tr>
<td>14</td>
<td>S2-4</td>
<td>The second plate with fibers 4 kg/m^3</td>
</tr>
<tr>
<td>15</td>
<td>S3-4</td>
<td>The third plate with fibers 4 kg/m^3</td>
</tr>
<tr>
<td>16</td>
<td>S1-5</td>
<td>The first plate with fibers 5 kg/m^3</td>
</tr>
<tr>
<td>17</td>
<td>S2-5</td>
<td>The second plate with fibers 5 kg/m^3</td>
</tr>
<tr>
<td>18</td>
<td>S3-5</td>
<td>The third plate with fibers 5 kg/m^3</td>
</tr>
</tbody>
</table>

3 RESULTS AND ANALYSIS

Tensile strength testing using the sides generally cylindrical specimen. When the load P reaches a maximum, tested concrete cylinders will be split. Split tensile strength is calculated as follows:

\[ f_{st} = \frac{2P}{\pi LD} \]  \hspace{1cm} (1)

Description:
- \( f_{st} \) is split tensile strength
- \( P \) is the maximum test load
- \( L \) is the length of the test object
- \( D \) is the diameter or width of the specimen

Generally split tensile strength of concrete value range (1/12 - 1/8) the compressive strength of concrete. Table 3 below shows the results of split tensile strength test on concrete cylinders 28 days.

Table 3. Results of tensile test of concrete cylinders split fiber and without fiber 28 days

<table>
<thead>
<tr>
<th>Age (days)</th>
<th>Specimen</th>
<th>( f_{st} ) (MPa)</th>
<th>( f_{st\ average} ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>28</td>
<td>C1-0</td>
<td>3.864</td>
<td></td>
</tr>
<tr>
<td>28</td>
<td>C2-0</td>
<td>3.949</td>
<td>3.954</td>
</tr>
<tr>
<td>28</td>
<td>C3-0</td>
<td>4.048</td>
<td></td>
</tr>
<tr>
<td>28</td>
<td>C1-4</td>
<td>3.114</td>
<td></td>
</tr>
<tr>
<td>28</td>
<td>C2-4</td>
<td>3.284</td>
<td>3.232</td>
</tr>
<tr>
<td>28</td>
<td>C3-4</td>
<td>3.298</td>
<td></td>
</tr>
<tr>
<td>28</td>
<td>C1-5</td>
<td>3.185</td>
<td></td>
</tr>
<tr>
<td>28</td>
<td>C2-5</td>
<td>3.326</td>
<td>3.279</td>
</tr>
<tr>
<td>28</td>
<td>C3-5</td>
<td>3.326</td>
<td></td>
</tr>
</tbody>
</table>
To test the bending of the plates will be determined the magnitude of the moment of cracking, bending strain, and deflection due to load. Plate bending test results without the use of fibers can be seen in Table 4.

<table>
<thead>
<tr>
<th>No.</th>
<th>Load x 10^3 (N)</th>
<th>strain x 10^-3</th>
<th>deflection mm</th>
<th>strain x 10^-3</th>
<th>deflection mm</th>
<th>strain x 10^-3</th>
<th>deflection mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>2</td>
<td>10</td>
<td>0.099</td>
<td>0.370</td>
<td>0.099</td>
<td>0.120</td>
<td>0.099</td>
<td>0.080</td>
</tr>
<tr>
<td>3</td>
<td>20</td>
<td>0.197</td>
<td>0.620</td>
<td>0.099</td>
<td>0.230</td>
<td>0.197</td>
<td>0.285</td>
</tr>
<tr>
<td>4</td>
<td>30</td>
<td>0.394</td>
<td>0.870</td>
<td>0.099</td>
<td>0.330</td>
<td>0.197</td>
<td>0.350</td>
</tr>
<tr>
<td>5</td>
<td>40</td>
<td>0.394</td>
<td>1.040</td>
<td>0.197</td>
<td>0.560</td>
<td>0.394</td>
<td>0.575</td>
</tr>
<tr>
<td>6</td>
<td>50</td>
<td>0.690</td>
<td>1.270</td>
<td>0.493</td>
<td>0.730</td>
<td>0.394</td>
<td>0.755</td>
</tr>
<tr>
<td>7</td>
<td>60</td>
<td>0.788</td>
<td>1.410</td>
<td>0.690</td>
<td>0.840</td>
<td>0.493</td>
<td>0.895</td>
</tr>
<tr>
<td>8</td>
<td>70</td>
<td>0.788</td>
<td>1.580</td>
<td>0.788</td>
<td>0.980</td>
<td>1.182</td>
<td>1.030</td>
</tr>
<tr>
<td>9</td>
<td>80</td>
<td>0.985</td>
<td>1.720</td>
<td>1.084</td>
<td>1.140</td>
<td>1.182</td>
<td>1.140</td>
</tr>
<tr>
<td>10</td>
<td>90</td>
<td>1.379</td>
<td>1.850</td>
<td>1.281</td>
<td>1.230</td>
<td>1.182</td>
<td>1.275</td>
</tr>
<tr>
<td>11</td>
<td>100</td>
<td>1.576</td>
<td>1.950</td>
<td>1.478</td>
<td>1.315</td>
<td>1.379</td>
<td>1.345</td>
</tr>
<tr>
<td>12</td>
<td>110</td>
<td>1.970</td>
<td>2.040</td>
<td>1.773</td>
<td>1.395</td>
<td>1.576</td>
<td>1.450</td>
</tr>
<tr>
<td>13</td>
<td>120</td>
<td>2.364</td>
<td>2.120</td>
<td>2.069</td>
<td>1.475</td>
<td>1.576</td>
<td>1.530</td>
</tr>
<tr>
<td>14</td>
<td>130</td>
<td>-</td>
<td>-</td>
<td>2.266</td>
<td>1.600</td>
<td>1.872</td>
<td>1.645</td>
</tr>
<tr>
<td>15</td>
<td>140</td>
<td>-</td>
<td>-</td>
<td>2.660</td>
<td>1.660</td>
<td>2.069</td>
<td>1.740</td>
</tr>
<tr>
<td>16</td>
<td>150</td>
<td>-</td>
<td>-</td>
<td>3.645</td>
<td>2.245</td>
<td>2.364</td>
<td>1.850</td>
</tr>
<tr>
<td>17</td>
<td>160</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2.758</td>
<td>1.940</td>
</tr>
<tr>
<td>18</td>
<td>170</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>3.152</td>
<td>2.025</td>
</tr>
</tbody>
</table>
Description:
For plates without the addition of fibers, cause cracks in the load of 130,000 N, while the composition of the fiber plate with 4 kg/m$^3$ and 5 kg/m$^3$ consecutive fractured at a load of 150,000 N and 170,000 N but still ductile.

Figure 4 and Figure 5 respectively show the relationship between the load vs strain and load vs deflection for a plate with thickness $t = 200$ mm.

Figure 4. Diagram of load vs. strain for plate with thickness $t = 200$ mm

Figure 5. Diagram of the load vs. deflection for plates with a thickness of 200 mm
Pattern collapse occurs during plate bending test is shown in Figure 6 below.

![Figure 6. Pattern collapse during bending test plate with thickness t = 200 mm](image)

Based on the test results, the magnitude of the maximum bending moment occurs can be determined by using the following formula:

$$M_{	ext{rentur}} = \sigma \times W$$  \hspace{1cm} (2)

**Description:**
- $M_{\text{rentur}}$ is plate bending moment (N.mm)
- $\sigma$ is the maximum stress (N/mm$^2$)
- $W$ is the static moment (mm$^3$)

4 DISCUSSION

For the split tensile strength testing, the value of the voltage ($f_u$) in Table 3 is obtained based on the calculation formula (1). Split tensile strength of concrete testing performed at 28 days. Characteristic compressive strength of 33 MPa cylinder is. In normal concrete (concrete without fibers), split tensile strength values the average of the third cylinder is 3, 954 MPa or approximately 1/10 of the compressive strength characteristics. Based on Table 3, for concrete fiber with the fiber composition of 4 kg per cubic meter of concrete mixture produces a strong dance values average 3.232 MPa. This value does not vary much with the value of the tensile strength of concrete with fiber composition of 5 kg per cubic meter of concrete mixture that is 3.279 MPa. In the case of split cylinder tensile strength, it can be concluded that the concrete without fiber has a value 20% greater capacity than concrete with fiber composition. Figure 7 shows the diagram comparison of split cylinder tensile strength of concrete at 28 days.

![Figure 7. Diagram comparison of tensile strength cylinder 28 days](image)
The behavior of the fiber concrete tensile strength test cylinders of different sides to the plate bending test. Table 4 shows the results of the bending test without fiber plate, and with the addition of each fiber to 4 kg/m³ and 5 kg/m³ to the concrete mix. From each of the three test specimens, taken one of the specimens considered to represent. Strain behavior of the plate when done loading different values between the plates without the use of fiber to fiber. Based on Table 4, the maximum load that can be borne without fiber plate with a strain of 120,000 N and 0.002364. At 130,000 N loading, the plate had cracked and broke immediately. From Figure 4, the plate with the fiber composition of 4 kg/m³ increase load capacity by 17% and increase ductility by 54% of the plate without fiber. While the composition of the plate with a 5 kg/m³ increase load capacity by 42% and increase ductility by 33% of the plate without fiber. The addition of fiber composition on the plate of 4 kg/m³ to 5 kg/m³ can increase the load capacity, from 150,000 N to 170,000 N. However, it can not improve the ductility of strain. Deflection behavior of the plate without the plate with the addition of fibers and fiber identical to the strain behavior.

Based on Figure 5, the addition of deflection at the plate with the fiber composition of 4 kg/m³ is 6% larger than the plate without fiber. From Table 4, there are anomalies in the results of tests in which the deflection occurs at the plate with a composition of 5 kg/m³ was 2.025 mm smaller than the plate without fibers (2.120 mm). This could be due to the mixing process of concrete mix with fiber is not homogeneous, so that the fibers do not spread evenly. The magnitude of the resulting bending moment can be seen in Table 5.

<table>
<thead>
<tr>
<th>No</th>
<th>Type</th>
<th>$\sigma_{\text{maximum}}$</th>
<th>W</th>
<th>$M_{\text{maximum}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Plate without fiber</td>
<td>0.120</td>
<td>20000000</td>
<td>2400000</td>
</tr>
<tr>
<td>2</td>
<td>Plate with the additional of fiber 4 kg/m³</td>
<td>0.150</td>
<td>20000000</td>
<td>3000000</td>
</tr>
<tr>
<td>3</td>
<td>Plate with the additional of fiber 5 kg/m³</td>
<td>0.170</td>
<td>20000000</td>
<td>3400000</td>
</tr>
</tbody>
</table>

From Table 5 plate bending moment magnitude obtained with the fiber composition of 5 kg/m³ is equal to 3400000 N.mm 42% greater than the plate without fiber.

5 CONCLUSION

Based on the above discussion, it can be concluded that the behavior of split cylinder tensile strength with flexural strength test on different plates. Significantly, the concrete cylinder specimen with the composition of the fiber can lower towing capacity divided by 22% compared to cylindrical concrete without fiber. However, the addition of fiber to the plate can increase the load capacity and ductility compared to plates without fiber. For it can be suggested that the use or the addition of fibers to the concrete mix is suitable applied to the construction element with a large surface area, such as road pavement, tunnels, and dams or docks.

6 REFERENCES

5. Product Data Sheet, ISO 9001 “Elasto Plastic Concrete” 2009
Identification and Mitigation of Risks during Design and Implementation of Output and Performance based Road Contracts

Author
Rowan Kyle

Position
Asset Manager and Technical Principal

Organization
Opus International Consultants

Country
New Zealand

Co-Author(s)

E-mail
Rowan.Kyle@opus.co.nz

Keywords:
Asset, Management, Mitigation, OPRC, Risk.

Abstract:
There is an increasing level interest in the implementation of the Output and Performance based Road Contract (OPRC) model as a means of introducing good road asset management practices around the world. This particular contract form provides governments, donors, and road authorities with a number of advantages compared to other maintenance contract models.

While there are several key factors which underpin the success of this form of contract, one of the critical elements is the identification and mitigation of key project risks involved with the concept development, implementation, and contract management phases. An early risk assessment will provide the delivery team an opportunity to identify and manage potential obstacles to success as soon as possible in the project cycle.

This paper will outline the steps involved in carrying out an adequate level of risk identification, the issues, and lessons that have been identified from previous OPRC projects, and the measures that are recommended to maximize the opportunity of project and contract outcome success.
Identification and Mitigation of Risks during Design and Implementation of Output and Performance based Road Contracts

Mr. Rowan Kyle\(^1\)

\(^1\)Opus International Consultants, Napier, NEW ZEALAND
Email for correspondence: Rowan.Kyle@opus.co.nz

1 INTRODUCTION

The adoption of performance based road contracts is recognised as a means of delivering good asset management in the transport sector (A Guide to Delivering Good Asset Management in the Road Sector through Performance Based Contracting 2014). An increasing number of Road Authorities (RA’s) administering networks around the world, both in developed and developing countries see benefits from implementing a modified form of design, build and maintain road contract model. The Output and Performance based Road Contract (OPRC) combines output (measure and value) and performance (lump sum) elements within a long term (typically 7 to 10 years) contract duration. The output components are used to manage the risks involved with Improvement (widening and other construction works undertaken to increase capacity), Rehabilitation (renewal of existing pavement strength), and Periodic Maintenance (surfacing renewal and any associated heavy maintenance) works. The performance based elements are then used to link compliance management of output component quality, as well as the delivery of the Routine Maintenance (e.g. pavement integrity, drainage, signs, delineation, vegetation etc.) service levels (World Bank Sample Bidding Documents 2006).

The factors driving the introduction of this model differ from country to country, however there are often a number of common issues, irrespective of the location or the stage of their road asset development, as listed below:

- The need for more stable and adequate funding for road maintenance from government, treasury, and/or international donors, especially following a period of rapid investment in developing the country’s road transport sector
- Frequent requests are made for increased funding allocations under existing force account or measure and value road maintenance contracts, but often with little linkage with the outcomes that are achieved
- Increasing political pressure applied to the RA to focus on customer’s (road users) needs, satisfaction, safety and/or value for money outcomes
- An awareness of the benefits of transferring more responsibility and risks to the contractor, to stimulate innovation and cost-effective delivery of resources
- Increased recognition that longer contract durations provide a higher level of certainty for investment in adequate plant and resources required for both construction and maintenance delivery and quality.

The extent of risk transfer under the OPRC model is outlined in the following Figure:

![Figure 1. Allocation of Risks across Various Road Maintenance Contract Models (Queirroz, 2011)](image_url)
2 CRITICAL STEPS IN THE DESIGN AND IMPLEMENTATION PROCESS

The strategy and inputs required to complete the design and the subsequent implementation of the OPRC model will depend upon where the country and RA sits on the risk spectrum illustrated in Figure 1. Furthermore there will be a range of unique political, institutional, financial, and physical factors that shape the way their road transport sector is being managed and operated. Many developing countries may be seeking to move from a Force Account operation to more of an outsourced model. Conversely some developed countries may see OPRC as a means of moving back from a fully performance specified contract model, thereby rebuilding a higher level of institutional knowledge and control over the management of their road assets. While the strategy may therefore vary in direction and detail, Greenwood and Kyle (2014) have illustrated that most, if not all, of following phases and steps will still be required.

![Figure 2: Critical Steps and Time Line for OPRC Design and Implementation (from Greenwood and Kyle 2014)](image)

The “Readiness Review” within the first phase of the process shown in Figure 2 must clearly identify the objectives in moving to an out-sourcing and performance based contract environment. Specifically this phase must confirm:

- The reasons and benefits for pursuing a reform of the existing road management processes
- The level of political support for the reform and management of the associated institutional changes required
- The potential impact of local or national election cycles on the timing of the implementation program
- The availability of sufficient long term funding to enable adequate renewal and maintenance investment
- The available institutional capacity within governmental and private sectors to manage the transition and contract delivery requirements
- The identification of any legislative, procedural, social, and/or environmental obstacles
- Other global risks to the procurement / implementation process, and whether these can be mitigated
- The assets to be included or excluded
- Availability of existing asset data and associated data management systems
- The existence of road networks suitable for transition into the OPRC model.
Experience to date indicates that this initial phase may take approximately six months to fully understand all of the related issues, but can take much longer where there is high level of uncertainty, particularly around the first five bullet points above. However where there is a high level of political support for reform, combined with a strong leadership within the RA, then this timeframe may be significantly reduced.

This first phase needs to be well structured and rigorously managed to ensure that any decision to proceed to the second phase is well founded and justified. Where this is not the case then there is a risk that obstacles will be encountered in subsequent phases of the project that will have the potential to significantly delay progress or even result in the failure of the project. It is therefore critical that these steps are led by experienced personnel from the right organization to ensure that the correct objectives are being pursued, and that there is very good alignment of these within and between the respective organisations.

Following a final decision to proceed, a detailed design and implementation program is necessary to clearly identify the inputs, resources, and timing of outputs required to achieve the agreed date for contract commencement. Many of the subsequent inputs and detail will be specific to the actual roads forming the contract package, and the confirmed selection of these will lie on the critical path of the project program. It is therefore strongly recommended that this road selection be agreed and fixed as early as possible, and that any subsequent additions or substitutions are avoided to minimise delays that will result from the need to collect further condition and design data. Other essential steps in this phase are summarised below.

i) Capacity Building Program

Once any weaknesses and/or deficiencies have been identified in either the employer’s organization and/or contracting industry, then a program of capacity building should be pursued. For example this would include the following activities:

- Developing or updating of required asset management systems including required installation, data collection, and training for the RA personnel administering these systems
- Hiring and training of asset management and contract management personnel dedicated to the project
- Workshops to raise awareness over the form, scope, operation and management of the OPRC model (pre and post contract award) for both the employer, contracting and local engineering consulting industry
- Monitoring and dissemination of the learnings to be included in future contracts.

Typically where the local contracting industry has little or no experience with the design/build concept, they will not have sufficient in-house expertise to effectively manage the design, routine maintenance programming, compliance auditing, and Right of Way (RoW) management components required under this contract model. Consequently most bidders will need to form a consortium comprised of an experienced contractor and a professional design consultant to provide the required resources, experience, and skills. The formation of this consortium needs to be robust and comprised of organizations that are culturally aligned to ensure a long-term sustainable partnership. Experience has shown that if a breakdown in this relationship occurs during the contract period significant difficulties can occur in the delivery and success of the contract as a whole.

ii) Road Network Selection

The selection of the roads must be made taking the following important issues into consideration:

- The roads must form a contiguous network so that they can be effectively and economically managed by the contractor, without excessive effort being spent either back tracking, or through a loss of productivity in terms of inspections and routine maintenance activities
- The geographic layout of the network should be selected so that ideally a contractor can establish a base, or depot, near the centre that minimises his response time to emergencies or incidents
- The network should be long enough to enable a contractor to have sufficient maintenance works to establish and sustain a work force on site for the full duration of the contract period, not just any initial construction phase
- The risks due to large scale geological instability, major regular flood events, future significant changes in traffic loading, or the impact of planned capital works programs (including the addition and/or subtraction of road sections), should either be avoided, or explicitly managed outside of the lump sum contract price.

iii) Development of RA Asset Management Capacity and Data Preparation Program

The effective sharing of risk by the contractor under the OPRC lump sum model requires a much higher level of inventory and condition data to be made available to bidders. Critical to this process is the development of the RA’s internal asset management systems and experience, which must be either in place, or implemented in parallel with the OPRC project. Experience has shown that the extent and accuracy of data which may be acceptable for traditional contracts, is unacceptable when the risk for the design, quantum of work and long term performance of
both construction and maintenance works is transferred to the contractor. This requirement in turn increases the focus of the asset management needs of the network, as both the employer and contractor become much more aware of the need for accurate and complete data sets for the management and operation of the contract. As the collection and analysis of inventory and condition data will be linked to the payments the contractor will ultimately receive, this information becomes of increasing interest to both parties.

iv) Contract Design and Bidding Documents

The bidding document structure and content for each project will be different, and will reflect the objectives behind the implementation of this contract model, the specific requirements of the road network, and the how far along the asset management pathway the particular RA may be at the outset. The following steps must be addressed in detail during this development phase:

- Identify, agree and specify appropriate service levels for the respective project roads ensuring they are aligned with the functions of the particular road classifications, affordable and not out of context with adjacent roads remaining outside of the OPRC contract
- Develop an appropriate legal, social and environmental framework reflecting the specific legislative requirements of the country, and the particular needs of the road network
- Review and agree an appropriate contract format, including the:
  - use of existing sample bidding documents and conditions of contract,
  - assets to be included and reasons for any that are excluded along with the responsibility for maintaining them,
  - performance measures to be adopted and compliance mechanisms to be used,
  - required residual pavement lives at the time of hand-back, and how will these be measured,
  - recommended procurement strategy, method, and evaluation procedures.
- The development of a process for the identification, rating, and allocation of risk
- The development of appropriate conceptual design detail for all Improvement, and Rehabilitation works including survey, drawings, specifications, safety requirements and codes
- The identification of possible emergency events and associated basis of payment to be implemented
- The development of a detailed cost estimates and contract specific financial model, necessary to identify the most appropriate contractor cost recovery (payment) model. In many situations there will be a high level of uncertainty over the quantity and/or rates used to determine the detailed cost estimate due to a lack of maintenance history records or input costs on a commercial basis. In these situations it is recommended that a form of risk modelling is used to examine the distribution in the cost estimate values particularly at the 5th, 50th (expected) and 95th percentile ranges.
- The spreading of an appropriate and sustainable percentage of profit from any initial Improvement or Rehabilitation phase across the Routine Maintenance phase (monthly lump sum payments). This is vital to ensure the contractor remains interested and incentivized to deliver the specified service levels over the entire contract duration.
- The selection of appropriate and adequate bonds or guarantees as additional protection instruments.

v) Procurement of the OPRC Monitoring Consultant

If the need for the engagement of a Monitoring Consultant has been identified during the initial “Readiness Review” phase to provide support and training to the employer, then the timing of this engagement will crucial to the success of the project. Experience has shown that earlier, rather than later, engagement provides benefits to the employer in terms of alignment during the bidding document preparation, as well as support during bid evaluation, contract award and contract start-up phases. Late or delayed engagement, especially well beyond the contractor start date, has been shown to introduce the following risks:

- The engagement of a short term consultant to administer the contract during the start-up and design phase becomes necessary
- The subsequent transition phase between consultants can result in miscommunication, differences in opinion over appropriate and acceptable design detail prepared by the contractor
- Delays in approving the detailed designs due to changing personnel and the need for the newly appointed monitoring consultant to revisit design information previously submitted by the contractor
- Delays in the provision of contractor supplied facilities, where a contractor may take advantage of a late appointment, leading to inefficiencies in inspections or monitoring due to increased travel time and reduced operational efficiency by the consultant.

It is recommended that the same consultant be engaged for both the preparation of the contract documents and the implementation phase. This approach not only mitigates the above risks, but also provides a much higher degree of continuity, ownership, and project knowledge of both the development and implementation phases, which in turn will be beneficial to the management of project as a whole.

vi) Procurement of the OPRC contractor

The procurement process (pre-qualification or no prequalification, single envelop, or two (separate price) envelop), and evaluation criteria to be used, must be agreed once the assessment of the contracting industry capacity has been completed. In most instances it is likely that both the RA, and the local contracting industry, will be unfamiliar with this form of contract, and therefore a series of workshops leading up to, and during, the bidding phase will be required to adequately explain the concepts, the level of risk sharing and basis of payment. A failure to complete this awareness raising prior to the bidding phase is likely to result in a lack of adequate bidders, and/or very high bid prices being received. In this situation the industry may perceive this form of contract to be far too risky, especially compared to traditional measure and value contract models which they are likely to be more familiar with.

The time to contract award will be dependent upon the approval of the bidding documents by the RA, the provision of a No Objection Certificate from the donor, the time allowed for bidding (typically a minimum of three months), and whether any issues or tags are encountered during the evaluation phase. It is essential that the procurement program makes sufficient allowance for the statutory approval timeframes, and the risk of an extended negotiation period being required if issues are encountered.

It is our experience that a two envelop approach, combined with a process of scoring the value of bidders’ technical proposals (quality), increases the probability of securing the best contractor. By opening the price envelops after the evaluation of the technical proposals removes the influence of the price on either acceptance or rejection of a bid. By adopting a weighted scoring mechanism that recognizes the value of both the technical proposal and price will assist with encouraging experienced and capable contractors to bid for these contracts. While country specific issues of probity and process transparency will still need to be addressed, we believe a move to develop and implement a quality/price evaluation method would be beneficial to the successful procurement of OPRC contractors in the future.

3 PROJECT RISK IDENTIFICATION PROCESS

Where possible a formal risk identification workshop early in the design phase should be run, with representatives of the RA, and the contracting industry invited to attend. While an initial set of generic risks can be provided to seed the discussion, it is recommended that this should be initially structured around a “brain storming” exercise without any constraints in an effort to identify all potential risks from a wide range of perspectives, irrespective of the probability of being actually encountered during the project duration.

Once all possible risks have been captured, then these need to be ranked by applying an agreed scoring criteria based upon both the consequence (C) and likelihood (L) of an identified risk event occurring. A qualitative score (R) can then be derived for each identified risk where:

\[ R = C \times L \]  \hspace{1cm} (1)

An example of some typical consequence and likelihood scoring criteria have been provided in the following tables (Downing 2013).
Table 1. Example Consequences (C) Matrix Impacting upon Road Maintenance and Operation.

<table>
<thead>
<tr>
<th>Consequence</th>
<th>Minimal</th>
<th>Minor</th>
<th>Moderate</th>
<th>Major</th>
<th>Substantial</th>
</tr>
</thead>
<tbody>
<tr>
<td>Category / Score</td>
<td>1</td>
<td>10</td>
<td>40</td>
<td>70</td>
<td>100</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Harm to People</th>
<th>Injury requiring short-term treatment / First Aid care</th>
<th>Injury requiring short-term medical treatment</th>
<th>Injury requiring extended treatment, or temporary incapacity</th>
<th>Injury requiring permanent disability or fewer than three deaths</th>
<th>Three or more deaths</th>
</tr>
</thead>
<tbody>
<tr>
<td>Financial / Legal Cost</td>
<td>Costs able to be absorbed within budget</td>
<td>$100k to &lt;$1m</td>
<td>$1m to &lt;$10m</td>
<td>$10m to &lt;$50m</td>
<td>$50m +</td>
</tr>
<tr>
<td>Environment Damage</td>
<td>Repaired within 7 days</td>
<td>Repaired within 4 weeks</td>
<td>Repaired within 1 - 5 months</td>
<td>Repaired within 6 - 12 months</td>
<td>Repair takes more than 1 year to restore or is permanent</td>
</tr>
<tr>
<td>Government / Ministerial Interest</td>
<td>Government/Ministerial enquiry that is successfully resolved</td>
<td>Negative feedback from Minister requiring executive response</td>
<td>On-going Parliamentary / ministerial questions</td>
<td>Potential for loss of ministerial confidence / formal enquiry by Government statutory agency</td>
<td>Loss in ministerial confidence / Commission of Inquiry / dismissal of Authority personnel</td>
</tr>
</tbody>
</table>

Table 2. Example Likelihood (L) Matrix Impacting upon Road Maintenance and Operation.

<table>
<thead>
<tr>
<th>Likelihood</th>
<th>Rare</th>
<th>Highly Unlikely</th>
<th>Unlikely</th>
<th>Possible</th>
<th>Likely</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description / Score</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>Frequency</td>
<td>Consequence may occur in exceptional circumstances (11+ years)</td>
<td>Consequence may occur in the next 6 - 10 years</td>
<td>Consequence may occur in the next 3 - 5 years</td>
<td>Consequence may occur in the next 1 - 2 years</td>
<td>Consequence may occur in the next 12 months</td>
</tr>
<tr>
<td>Probability</td>
<td>Probability &lt; 1%</td>
<td>Probability 1% - 9%</td>
<td>Probability 10% - 19%</td>
<td>Probability 20% - 50%</td>
<td>Probability greater than 50%</td>
</tr>
</tbody>
</table>

By applying the highest score calculated from the Consequence and Likelihood matrix tables, each of the identified risks can then be ranked based upon the value of the risk score (R). An example of a risk score prioritization matrix is provided below.

Table 3. Risk Score (R) Prioritization Score Matrix.

<table>
<thead>
<tr>
<th>Risk Score (R)</th>
<th>Likelihood</th>
</tr>
</thead>
<tbody>
<tr>
<td>Consequence</td>
<td>Rare</td>
</tr>
<tr>
<td>Substantial</td>
<td>100</td>
</tr>
<tr>
<td>Major</td>
<td>70</td>
</tr>
<tr>
<td>Moderate</td>
<td>40</td>
</tr>
<tr>
<td>Minor</td>
<td>10</td>
</tr>
<tr>
<td>Minimal</td>
<td>1</td>
</tr>
</tbody>
</table>

Score Priority | Very Low | Low | Moderate | High | Very High |

Once all identified risks have been prioritized, the next step is to then develop appropriate mitigation strategies and plans only for those risks ranked as either high or very high. The decision on what level of priority should, or should not, be given further consideration needs to be jointly agreed by all parties involved in the risk identification and ranking exercise.

It is recommended that a process of regular reviews is undertaken throughout the design and implementation phases, to monitor the impact of any changes to the identified risks, and prioritize any new risks that have arisen.
Even after contract award, it is advisable that both the employer and contractor maintain this process in place as a formal way of managing and reporting specific contract risks.

Even with the development of specific risk mitigation plans there may still be a residual level of risk remaining in some situations that cannot be further mitigated, along with the possibility of other unexpected risks suddenly emerging. These can never be entirely eliminated, and can only be effectively managed through the maintenance of very good levels of communication, clear lines of responsibility for decision making, and adequate contingency planning. Where these processes are in place the ability for an organisation to respond to, and successfully manage, these unforeseen risks will be strengthened.

4 SUMMARY OF TYPICAL PROJECT RISKS AND ALLOCATION WITHIN THE CONTRACT

Following the completion of the risk identification and prioritisation steps, this information can then be used to determine those risks that need to be effectively shared between the employer and contractor within the OPRC model. The General Conditions within the Sample Bidding documents already define some of the risks carried by the employer, such as the impact of war, rebellion, forces of nature etc., with all remaining risks then assigned to the contractor. However this general assignment of liabilities is insufficient to effectively manage the wide range of other risks which will be shared between the two parties, on the basis that they should be assigned to the party best able to manage them.

Under the OPRC model there is a clear transfer of risk from the employer to the contractor through the lump sum price model and the basis of payment. The extent of this transfer, and what if any limits are placed on this from the contractor’s perspective, should be carefully worked through jointly by the employer and representatives of the contracting industry before the bidding phase commences. This would then allow any potential problems or concerns to be identified, and addressed along with improving the understanding over the nature and extent of the risks to be shared.

Experience to date has indicated that around 50 to 60 specific risks may need to be carefully considered, and the responsibility for them clearly defined. Once jointly agreed, they should then be summarised within a table and included under an appendix in the bidding documents. This tabular summary is termed the contract “Risk Profile” and becomes contractually binding. However, should a situation arise over these long term contracts that makes it necessary for some adjustment to this profile to be made, then so long as both parties agree to this change, it could be amended as a variation to the contract. In these circumstances neither party should un-necessarily withhold agreement, where such a change would be beneficial to the way the road network is managed from the perspective of the contractor, employer, and road users.

The following table provides an example of several typical risks included with an OPRC Risk Profile. It firstly describes the specific risk, the party that carries this risk, and what limit or boundary applies to those transferred to the contractor, beyond which the liability will then revert back to the employer.

Table 4. Example of Several Contract Risk Profile Items

<table>
<thead>
<tr>
<th>Risk Description</th>
<th>Contractor Risk</th>
<th>Employer Risk</th>
<th>Risk Boundary</th>
</tr>
</thead>
<tbody>
<tr>
<td>Changes to the road network length, except as provided for in the contract documents</td>
<td></td>
<td>✓</td>
<td>Intractable landowner issues</td>
</tr>
<tr>
<td>Maintaining private access-ways/pedestrian facilities in defined urban or built-up areas.</td>
<td></td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>Identification and management of road aggregate stockpile and waste material disposal sites</td>
<td>✓</td>
<td></td>
<td>Provision of an unencumbered RoW</td>
</tr>
<tr>
<td>Identification and reporting to the employer of the need for any specific clearances required for improvement and/or safety works</td>
<td>✓</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seeking and obtaining approval for other unforeseen RoW clearances</td>
<td></td>
<td>✓</td>
<td></td>
</tr>
</tbody>
</table>
5 CONCLUSIONS AND RECOMMENDATIONS

The adequate identification, prioritization, and mitigation of risks underpin the success of any project and should be considered as good business management. Unfortunately many project personnel either fail to recognize potential risks or take an overly optimistic view (optimism bias) when making ad-hoc assessments of the implications of those risks that are known.

The successful design and implementation of the OPRC model for long term maintenance of road networks will require careful consideration to be given to a very wide range of critical issues. This may need to be undertaken within a governmental and/or institutional environment that has both little experience of this form of lump sum / design build approach, and which itself may be undergoing rapid and significant change. Under these circumstances the opportunity for un-identified issues to become obstacles to, and/or undermine the successful achievement of, the projects objectives should not be underestimated.

The critical steps in establishing an effective risk mitigation process are considered to be:

- The preparation of a detailed program for the “Readiness Review” phase, with a clear decision tree developed and used for determining a “Go” or “No Go” for the next phase of the project
- The appointment of an experienced and dedicated Project Director / Manager, and the individual provided with a high level of political support, to manage the development and implementation phase
- Ensuring there is continuity in the experienced project personnel appointed by the RA to manage the project through to, and preferable beyond, the contract award date
- The preparation of a detailed program for the development and implementation phase that makes sufficient allowance for reviews and approvals by external agencies and donors
- Risk identification and planning is undertaken as early as possible in the “Readiness Review” phase and this is monitored and updated on a regular basis throughout the entire duration of the project.

In summary the steps, and processes outlined in this paper have been provided as a guide to managing the risks associated with the design and implementation of the OPRC model. They are however equally applicable to other contract forms, and if incorporated into on-going business practices, the probability of achieving successful project outcomes will be significantly increased.

6 ACKNOWLEDGEMENTS

The assistance provided by Dr Ian Greenwood in the development of the program shown in Figure 2 is acknowledged and has been extracted from a draft “Guideline for the Introduction of OPRC”, which is currently under preparation.

7 REFERENCES


PAPER TITLE
(90 Characters Max)
Benefit Evaluation of Road Rehabilitation at Nine Provinces in Indonesia

TRACK

AUTHOR
(Capitalize Family Name)
Tonny Judiantono

POSITION
Lecture

ORGANIZATION
City and Regional Planning Program, Faculty of Engineering – Bandung Islamic University

COUNTRY
Bandung, West Java, Indonesia

CO-AUTHOR(S)
(Capitalize Family Name)

POSITION

ORGANIZATION

COUNTRY

E-MAIL
(for correspondence)
tjudiantono@yahoo.com

KEYWORDS:
Road Rehabilitation, Project Evaluation, Direct Benefit, Indirect Benefit

ABSTRACT:
All of project development needs to be evaluated how far that project will gives benefit. Benefit of a project can be form direct benefit or indirect benefit. Road rehabilitation is an action to improve road performance. As result of road rehabilitation is increasing of IRI, road capacity, actual road design speed, and direct benefit rehabilitation have the shape of reduce of Vehicle Operating Cost (VOC), reduce in travel time and efficiency on maintenance cost of the road. Indirect benefit of road rehabilitation has effect on physical, social, economic, environmental and spatial, as like economic increase, people welfare increase, works opportunities, population growth acceleration, migration growth, social statues increase, changes on product distribution of commodities, loading-unloading, regional development, land use distribution, wide land production especially for agriculture, and changes on accessibility level. This research takes road projects between 1990-1998 in Bengkulu, South Sumatera, Lampung and West Java Provinces. Through direct observation on sample data and simple analysis by multiregression method, correlation method and comparative method between before and after condition of the road rehabilitation project, has result IRI changes up to 140% in average more than condition before rehabilitation, ADT increase 140%. For Lampung and Bengkulu up to 150% - 165%, West Java and Lampung increase 360%-470%. This study also resulting the relation between Speed and IRI such as: Speed = -0.13443*IRI+49.71993 (R=0.98)

Vehicle composition will changes also especially on car and utility, meanwhile truck and bus tend to decline. After rehabilitation VOC decrease 21-46% average, and BCR 4.38 with benefit around 30 billion rupiahs each link for periode 1990 to 2010. Beside of benefit, the road rehabilitation raise negative impact as like level of traffic accident, productive land for food and crops planted which changes to industrial, residential or other non-agriculture.
Benefit Evaluation of Road Rehabilitation
At Nine Provinces in Indonesia

Tonny Judiantono

City and Regional Planning Program, Faculty of Engineering – Bandung Islamic University
Email: tjudiantono@yahoo.com

1. INTRODUCTION

Research Background

Every project development needs to be evaluated how far that project gives benefit. Benefit of a project can be evaluated by direct benefit and indirect benefit. Rehabilitation is an action to improve road performance which result increase on IRI, road capacity, actual road design speed, and all of that gives direct benefit in the shape of reduce on Vehicle Operating Cost (VOC), reduce in travel time and efficiency on road maintenance cost. It gives also indirect benefit in the form of effect on regional development, economic increase, people welfare increase and works opportunities.

Success criteria of the project development can be seen through the changing of indicators before and after project. It show by construction services indicators, traffic indicators and social-economic indicators. That indicators draw by the changes of IRI (International Roughness Index), numbers of vehicles (by Traffic Counting-TC), Vehicle Operating Cost (VOC), and social economic gain base on EIRR etc.

Thus evaluation is essentially to obtain information of roads and bridges rehabilitation benefit, are it equal with investment value or not?

Research Objectives

Objectives of this research are:
1. To evaluate the benefit of road and bridge rehabilitation project especially on:
   • Road and bridge constructions level of services.
   • Traffic level of services.
   • Social economic level of services.
2. Resulting simple and sharp method for measuring road rehabilitation benefit.
3. Gives recommendation to whose concern with this road project rehabilitation.

Scope of Research

Scope of this research is:
• To identify sample link at four of nine provinces which will become project representatives of the provinces. See Table 1 List of Link will be surveyed, as road sample group.
• Collecting data of road condition from samples group, complete with photos.
• Collecting traffic counting data from Dinas Binamarga each Provinces for link sample
• To collect all of road work information including financial cost, sources of finance, since that road has rehabilitated.
• To collect socio-economic data before and after rehabilitation.
• Collecting data from Sub Direccion of Administrasi Bantuan Luar Negeri, Proyek Peningkatan dan Pengawasan Teknik Peningkatan Jalan (Road Betterment Office - RBO), Dinas Pekerjaan Umum Bina Marga of the Province where the project are.
• Analyzing direct benefit and indirect benefit of that road rehabilitation projects.
• Conclusion and recommendation for the next project.

Table 1 List of Link to be surveyed

<table>
<thead>
<tr>
<th>Province</th>
<th>Package No.</th>
<th>Link No.</th>
<th>From ... To ...</th>
<th>Based on Fields KM-POST</th>
<th>Project Cost Rp</th>
<th>Time of Construction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>From Km.</td>
<td>To Km.</td>
<td>Length (Km)</td>
</tr>
<tr>
<td>Bengkulu</td>
<td>G-23</td>
<td>13.002</td>
<td>Kepahyang-Curup</td>
<td>61.000</td>
<td>85.000</td>
<td>24.000</td>
</tr>
<tr>
<td></td>
<td>N-13</td>
<td>13.008/1</td>
<td>Curup-Km 127</td>
<td>85.000</td>
<td>127.000</td>
<td>42.000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>13.008/2</td>
<td>Km 127 - Mr.Aman/T.Sawah</td>
<td>127.000</td>
<td>172.000</td>
<td>45.000</td>
</tr>
<tr>
<td>N-14</td>
<td>13.006/1</td>
<td>Manna - Tanjung Kemuning</td>
<td>143.000</td>
<td>184.300</td>
<td>41.300</td>
<td>12,108,025,029</td>
</tr>
<tr>
<td></td>
<td>13.006/2</td>
<td>Tanjung Kemuning - Linau</td>
<td>184.300</td>
<td>230.000</td>
<td>45.700</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
2. RESEARCH METHODOLOGY

Framework of this research can be seen on figure 1, and Steps of Research showed on figure 2.

![Figure 1 Approach Framework](image)

![Figure 2 Steps of Research](image)
3. RESULT ANALYSIS

Base on the references, Benefit Indicator of Road rehabilitation can be showed by Some indicators will be use for valuation that beneficiary, that are:

A. Indicator of transportation and roadway benefit
   1. Changes on IRI
   2. Changes on Traffic volume (ADT)
   3. Changes on speed
   4. Changes on vehicle composition
   5. Changes on Vehicle Operating Cost (VOC)
   6. Changes on maintenance cost
   7. Benefit cost analysis (base on VOC and maintenance cost) and EIRR of the roadway.

B. Indicator of Social changes benefit
   1. Changes on population growth acceleration
   2. Changes on migration acceleration
   3. Social statues changes

C. Indicator of Economic changes benefit
   1. Changes on GRDP growth acceleration
   2. Changes on production distribution for each commodity
   3. Changes on level of load and unload handling

D. Indicator on physical and environmental physics benefit
   1. Changes on land distribution (land use pattern)
   2. Changes on productive land
   3. Changes on accessibilty

A. Transportation and roadway benefit

1. Changes on IRI
   Base on survey, and IRI data evaluation for condition before and after project at 1998, can be seen on table 2.

<table>
<thead>
<tr>
<th>NO</th>
<th>LOCATION</th>
<th>DATA EV.</th>
<th>SURVEY</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>ADT</td>
<td>IRI</td>
</tr>
<tr>
<td>1</td>
<td>Bengkulu</td>
<td>46.58</td>
<td>150.75</td>
</tr>
<tr>
<td>2</td>
<td>Sumatera Selatan</td>
<td>222.37</td>
<td>115.71</td>
</tr>
<tr>
<td>3</td>
<td>Lampung</td>
<td>98.86</td>
<td>165.05</td>
</tr>
<tr>
<td>4</td>
<td>Jawa Barat</td>
<td>150.36</td>
<td>135.00</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>129.54</td>
<td>141.63</td>
</tr>
</tbody>
</table>

Source: Survey, 2004

2. Changes on Average Daily Traffic (ADT)
   Base on survey and Binamarga data analysis, the increasing of IRI will push the increasing on ADT, especially for South Sumatera, Lampung and West Java, which increase more than 3 times after rehabilitation.
   Table 2 also shown the increasing of ADT, more influenced by population and GRDP growth. So by using multi regression approach can be forecast the future of ADT, with numbers of population and GRDP as independent variable. The relation between IRI and ADT before and after project see Figure 3.

Figure 3 ADT and IRI Improvement
3. Changes on speed
   One of road rehabilitation objective is increasing the travel speed. Result survey at some roads links show the increasing of IRI automatically make increase on speed (travel speed). See Table 3, Table 4 and Figure 4.

Figure 4 The Relationship between Speed & IRI

Table 3 Relationship between IRI and Travel Speed

<table>
<thead>
<tr>
<th>IRI</th>
<th>Speed</th>
<th>IRI</th>
<th>Speed</th>
<th>IRI</th>
<th>Speed</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.42</td>
<td>45.00</td>
<td>7.17</td>
<td>43.71</td>
<td>4.23</td>
<td>57.86</td>
</tr>
<tr>
<td>6.42</td>
<td>44.12</td>
<td>5.78</td>
<td>56.32</td>
<td>6.52</td>
<td>55.38</td>
</tr>
<tr>
<td>6.42</td>
<td>38.12</td>
<td>5.34</td>
<td>48.86</td>
<td>10.2</td>
<td>55.71</td>
</tr>
<tr>
<td>6.88</td>
<td>37.09</td>
<td>6.70</td>
<td>53.25</td>
<td>7.79</td>
<td>60.00</td>
</tr>
<tr>
<td>5.66</td>
<td>45.35</td>
<td>6.70</td>
<td>48.14</td>
<td>6.59</td>
<td>43.20</td>
</tr>
<tr>
<td>5.83</td>
<td>43.06</td>
<td>5.34</td>
<td>57.22</td>
<td>6.60</td>
<td>51.43</td>
</tr>
<tr>
<td>5.28</td>
<td>45.00</td>
<td>5.34</td>
<td>55.50</td>
<td>7.44</td>
<td>41.67</td>
</tr>
</tbody>
</table>

Source: Survey, 2004

Base on that data can be made a relationship formula between IRI and Travel Speed. The formula as below:

\[
\text{Speed} = -0.13443 \times \text{IRI} + 49.7193 \\
R = 0.98
\]

Base on that formula, can be shown the forecast relation between IRI and Travel speed as like Table 4

Table 4 Relationship IRI and Travel Speed (forecast)

<table>
<thead>
<tr>
<th>IRI</th>
<th>13.00</th>
<th>12.30</th>
<th>12.00</th>
<th>11.00</th>
<th>10.00</th>
<th>9.00</th>
<th>8.20</th>
<th>5.34</th>
<th>4.70</th>
<th>4.00</th>
<th>3.00</th>
<th>2.00</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPEED</td>
<td>47.97</td>
<td>48.07</td>
<td>48.11</td>
<td>48.24</td>
<td>48.37</td>
<td>48.51</td>
<td>48.62</td>
<td>49.00</td>
<td>49.09</td>
<td>49.18</td>
<td>49.32</td>
<td>49.45</td>
</tr>
</tbody>
</table>

4. Changes on vehicle composition
   Table 5 shows the changes of vehicle composition before and after project. It show the numbers of sedan, passenger car and mini-bus which it ordinary private vehicle increased in numbers, but for bus and truck which it ordinary commercial vehicle tend to decrease, except in the links at South Sumatera, Lampung and Java. Meanwhile the road links in P. Bangka and Bengkulu only private car has increased. So private car get more benefit from this rehabilitation project, meanwhile commercial car only a little bit get benefit from this road improvement.

5. Changes on Vehicle Operating Cost (VOC)
   In this research did not deed special case to calculate the changes on Vehicle Operating Cost (VOC) caused by the changes on IRI (as road rehabilitation result). For next benefit calculation of rehabilitation, we can use the standard formula Road User Cost model by Hoff & Overgaard (May 1992). See Table 6.

Table 5 Vehicle Composition Changes Before and After Project

<table>
<thead>
<tr>
<th>No.</th>
<th>PROVINCE</th>
<th>LINK No.</th>
<th>Year</th>
<th>Sedan</th>
<th>Pickup</th>
<th>Pax</th>
<th>Combi M.Bus</th>
<th>Bus</th>
<th>Truck</th>
<th>Heavy Truck</th>
<th>ADT</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>BENGKULU</td>
<td>130081</td>
<td>Before</td>
<td>0.113</td>
<td>0.553</td>
<td>0.158</td>
<td>0.055</td>
<td>0.116</td>
<td>0.005</td>
<td>819</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>0.092</td>
<td>0.663</td>
<td>0.104</td>
<td>0.039</td>
<td>0.100</td>
<td>0.001</td>
<td>797</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>BENGKULU</td>
<td>130082</td>
<td>Before</td>
<td>0.113</td>
<td>0.553</td>
<td>0.158</td>
<td>0.055</td>
<td>0.116</td>
<td>0.005</td>
<td>819</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>0.092</td>
<td>0.663</td>
<td>0.104</td>
<td>0.039</td>
<td>0.100</td>
<td>0.001</td>
<td>797</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>BENGKULU</td>
<td>130061</td>
<td>Before</td>
<td>0.115</td>
<td>0.484</td>
<td>0.130</td>
<td>0.064</td>
<td>0.204</td>
<td>0.002</td>
<td>1123</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>0.139</td>
<td>0.476</td>
<td>0.173</td>
<td>0.062</td>
<td>0.140</td>
<td>0.009</td>
<td>1771</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>BENGKULU</td>
<td>130062</td>
<td>Before</td>
<td>0.115</td>
<td>0.484</td>
<td>0.130</td>
<td>0.064</td>
<td>0.204</td>
<td>0.002</td>
<td>1123</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>0.139</td>
<td>0.476</td>
<td>0.173</td>
<td>0.062</td>
<td>0.140</td>
<td>0.009</td>
<td>1771</td>
<td></td>
</tr>
</tbody>
</table>
Table 6 Coefficient for Vehicle Operating Cost Formula (Rupiahs per Vehicle Km)

<table>
<thead>
<tr>
<th>Vehicle</th>
<th>k1</th>
<th>k2</th>
<th>k3</th>
<th>k4</th>
<th>k5</th>
<th>Base Cost</th>
<th>R2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Car</td>
<td>0.6838</td>
<td>24,851</td>
<td>0.00000252</td>
<td>0.0001050</td>
<td>0.001737</td>
<td>254.43</td>
<td>0.99</td>
</tr>
<tr>
<td>Utility</td>
<td>0.5547</td>
<td>28,008</td>
<td>0.00000927</td>
<td>0.0001410</td>
<td>0.001262</td>
<td>205.85</td>
<td>0.99</td>
</tr>
<tr>
<td>Small Bus</td>
<td>0.4872</td>
<td>28,078</td>
<td>0.00001110</td>
<td>0.0003440</td>
<td>0.000332</td>
<td>248.53</td>
<td>0.99</td>
</tr>
<tr>
<td>Large Bus</td>
<td>0.5807</td>
<td>20,159</td>
<td>0.00002140</td>
<td>0.0007085</td>
<td>0.002008</td>
<td>391.54</td>
<td>0.99</td>
</tr>
<tr>
<td>Light Truck</td>
<td>0.5422</td>
<td>24,086</td>
<td>0.00000956</td>
<td>0.0003420</td>
<td>0.000763</td>
<td>282.36</td>
<td>0.99</td>
</tr>
<tr>
<td>Medium</td>
<td>0.5049</td>
<td>20,612</td>
<td>0.00002350</td>
<td>0.0003660</td>
<td>0.000728</td>
<td>387.25</td>
<td>0.99</td>
</tr>
<tr>
<td>Heavy Truck</td>
<td>0.5603</td>
<td>16,601</td>
<td>0.00002290</td>
<td>0.0004070</td>
<td>0.000687</td>
<td>521.50</td>
<td>0.99</td>
</tr>
</tbody>
</table>

Source: Road User Cost model, Hoff & Overgaard, May 1992

Base on the above observation we can compare the influent of IRI on VOC before and after road rehabilitation. See Figure 7

![Figure 7 Relationship IRI & VOC](image)

6. Changes on maintenance cost

It is reasonable, road rehabilitation will reduce maintenance cost, in vice versa if the road does not rehabilitate will needs maintenance cost more over. Base on survey has been done, it show the maintenance cost before rehabilitation approximate Rp. 6.350,000,- each km/ year, meanwhile the maintenance cost after rehabilitation average Rp.4.100,000,- each km/year. In the next estimation, maintenance cost for those roads will increase 10% /year. For P.Bangka where it is separate island, maintenance cost tend higher than others. Detail of maintenance cost can see at Table 7 below:
7. Benefit Cost Analysis, NPV and EIRR of the road way
   From the financial side can be seen the changes on maintenance cost and vehicle operating cost (VOC). By calculate both cost component, the NPV and EIRR of the roads has been survey, for next 20 years since project has done, that is 1990-2010 period, with assume 2 times improvement must be done at this period, show NPV and EIRR are low.

   Table 7 Maintenance Cost (km/year)

<table>
<thead>
<tr>
<th>NO</th>
<th>TYPE</th>
<th>BEFORE (Rp)</th>
<th>AFTER (Rp)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sumatara and Java</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Shoulder</td>
<td>1,600,000</td>
<td>1,600,000</td>
</tr>
<tr>
<td>2</td>
<td>Roadway</td>
<td>3,500,000</td>
<td>1,250,000</td>
</tr>
<tr>
<td>3</td>
<td>Road sign</td>
<td>750,000</td>
<td>750,000</td>
</tr>
<tr>
<td>4</td>
<td>Bridge</td>
<td>500,000</td>
<td>500,000</td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>6,350,000</td>
<td>4,100,000</td>
</tr>
<tr>
<td></td>
<td>P.Bangka</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Shoulder</td>
<td>2,000,000</td>
<td>2,000,000</td>
</tr>
<tr>
<td>2</td>
<td>Roadway</td>
<td>4,500,000</td>
<td>1,750,000</td>
</tr>
<tr>
<td>3</td>
<td>Road sign</td>
<td>1,000,000</td>
<td>1,050,000</td>
</tr>
<tr>
<td>4</td>
<td>Bridge</td>
<td>750,000</td>
<td>750,000</td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>8,250,000</td>
<td>5,500,000</td>
</tr>
</tbody>
</table>

It is caused under uses of the roads, below its own capacity. The under uses of the roads is no more change since rehabilitation. So it occurs more caused just under utility factor, not causes the road rehabilitation is not utilize. The BCR, NPV and IRR for all roads show high point, but some roads show negative. It must not causes the road rehabilitation project is low enough, that is 2,6% / year and Gross periods 8 years, for 30 years loan.

Loan condition relatively soft, it means for infrastructure development like this is profitable. Yet because the new road rehabilitation is under utilize, not all of it capacity has used so the benefit which we get is low too, whereas for other roads receive positive result. See table 8

8. Changes on Road and Bridge capacity
   Another important manner of the roads rehabilitation is the changes on road and bridge capacity. Base on observation road capacity will increase between 7-14% after rehabilitation, even for South Sumatera some roads increase 40-60% up to before rehabilitation. For clearly see Table 9 below:

Table 8 Benefit of Each Road

<table>
<thead>
<tr>
<th>PROVINCE</th>
<th>LINK</th>
<th>BENEFIT</th>
<th>BCR</th>
<th>NPV (Rp)</th>
<th>IRR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bengkulu</td>
<td>13.006/1</td>
<td>18,187,318,602</td>
<td>2.1178</td>
<td>16,715,416,335</td>
<td>24.60%</td>
</tr>
<tr>
<td></td>
<td>13.006/2</td>
<td>18,187,318,602</td>
<td>3.2898</td>
<td>21,230,285,404</td>
<td>8.96%</td>
</tr>
<tr>
<td></td>
<td>13.008/1</td>
<td>18,187,318,602</td>
<td>1.5360</td>
<td>6,493,691,771</td>
<td>52.58%</td>
</tr>
<tr>
<td></td>
<td>13.008/2</td>
<td>18,187,318,602</td>
<td>1.5471</td>
<td>7,104,548,383</td>
<td>53.69%</td>
</tr>
<tr>
<td>South Sumatera</td>
<td>15.016</td>
<td>23,300,073,986</td>
<td>1.4930</td>
<td>16,715,416,335</td>
<td>24.60%</td>
</tr>
<tr>
<td></td>
<td>15.021/1</td>
<td>33,563,861,417</td>
<td>4.2438</td>
<td>24,217,609,960</td>
<td>51.59%</td>
</tr>
<tr>
<td></td>
<td>15.021/2</td>
<td>51,052,228,204</td>
<td>4.9883</td>
<td>37,150,185,008</td>
<td>62.47%</td>
</tr>
<tr>
<td></td>
<td>15.023</td>
<td>6,840,112,297</td>
<td>2.2116</td>
<td>4,430,727,665</td>
<td>19.22%</td>
</tr>
<tr>
<td></td>
<td>15.058</td>
<td>39,125,149,578</td>
<td>5.4081</td>
<td>27,923,287,710</td>
<td>70.89%</td>
</tr>
<tr>
<td></td>
<td>15.059</td>
<td>7,420,307,012</td>
<td>1.5465</td>
<td>4,143,038,188</td>
<td>9.85%</td>
</tr>
<tr>
<td></td>
<td>15.060</td>
<td>10,475,861,616</td>
<td>4.0349</td>
<td>7,671,760,483</td>
<td>80.23%</td>
</tr>
<tr>
<td></td>
<td>15.061</td>
<td>29,087,590,428</td>
<td>2.3305</td>
<td>20,320,312,802</td>
<td>32.05%</td>
</tr>
<tr>
<td>Lampung</td>
<td>17.002</td>
<td>58,879,961,667</td>
<td>10.0445</td>
<td>41,848,978,916</td>
<td>106.89%</td>
</tr>
<tr>
<td></td>
<td>17.003</td>
<td>29,329,337,705</td>
<td>10.0445</td>
<td>20,758,005,063</td>
<td>106.39%</td>
</tr>
</tbody>
</table>
|           | 17.004 | 38,128,791,100 | 10.0445 | 27,600,267,536 | 129.43%
|           | 17.048 | 13,692,680,199 | 10.0445 | 9,675,886,653  | 31.32% |
| West Java | 22.046/2 | 18,200,892,994 | 1.6543 | 11,654,916,777 | 16.49% |
|           | 22.070 | 47,614,318,694  | 3.7019 | 33,988,157,736  | 52.31% |
|           | 22.073 | 18,690,713,668  | 3.1212 | 13,051,474,796  | 39.82% |

Table 9 Change of Road Capacity

| No. | PROVINCE | LINK  | CAPACITY (smp/jam) | INCREASE (%)
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>BEFORE</td>
<td>AFTER</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
| 1   | BENGKULU | 130081 | 1,013               | 1234          | 121.81%
|     |          | 130082 | 1,013               | 1234          | 121.81%
|     |          | 130061 | 877                 | 1092          | 124.52%
|     |          | 130062 | 877                 | 1092          | 124.52%
| 2   | SOUTH    | 15,016 | 884                 | 1414          | 159.94%
|     | SUMATERA | 15,021 | 835                 | 864           | 103.45%
|     |          | 15,021 | 983                 | 1405          | 142.98%
|     |          | 15,023 | 858                 | 1012          | 117.92%
|     |          | 15,058 | 826                 | 1002          | 121.22%
|     |          | 15,059 | 723                 | 824           | 113.84%
|     |          | 15,061 | 685                 | 750           | 109.38%
|     |          | 15,061 | 721                 | 856           | 118.74%
| 3   | LAMPUNG  | 17,002 | .611                | 685           | 112.02%
|     |          | 17,003 | .611                | 1337          | 155.14%
|     |          | 17,004 | .782                | 841           | 107.52%
|     |          | 17,048 | .939                | 1015          | 108.04%
| 4   | WEST JAVA| 22,046 | 950                 | 1016          | 106.96%
|     |          | 22,07 | 842                 | 922           | 109.57%
|     |          | 22,073 | 774                 | 1178          | 152.23%
B. Social changes benefit
1. Changes on population growth acceleration
Through IRI increases after rehabilitation project, the ADT at that road will improve. The increasing of ADT will push faster immigration and outmigration of the citizen. That condition will causes added/ subtracted population faster, off course that added or subtract is not causes by natural increase of the population. Actually it often when the numbers of population increase, GRDP increase too at the same time with IRI increase after that rehabilitation project. The relationship between IRI, ADT and Population growth can be seen at table 10 and graph below.

Table 10 ADT, IRI and Population Increase

<table>
<thead>
<tr>
<th>No</th>
<th>PROVINCE</th>
<th>LINK NO</th>
<th>PENINGKATAN</th>
<th>ADT</th>
<th>IRI</th>
<th>PENDUDUK</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>BENGKULU</td>
<td>13.0081</td>
<td>0.97</td>
<td>2.51</td>
<td>1.09</td>
<td></td>
</tr>
<tr>
<td></td>
<td>13.0082</td>
<td>0.97</td>
<td>3.54</td>
<td>1.09</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>130.061</td>
<td>1.58</td>
<td>2.04</td>
<td>1.09</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>130062</td>
<td>1.58</td>
<td>2.28</td>
<td>1.09</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>SOUTH</td>
<td>15.016</td>
<td>0.87</td>
<td>1.93</td>
<td>1.13</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SUMATERA</td>
<td>150211</td>
<td>1.89</td>
<td>2.95</td>
<td>1.05</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>150212</td>
<td>1.89</td>
<td>2.28</td>
<td>1.05</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>15023</td>
<td>0.66</td>
<td>1.74</td>
<td>1.05</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>15.058</td>
<td>1.13</td>
<td>2.68</td>
<td>1.05</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>15.059</td>
<td>0.67</td>
<td>2.21</td>
<td>1.05</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>15.06</td>
<td>0.79</td>
<td>1.21</td>
<td>1.05</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>15.061</td>
<td>0.76</td>
<td>2.24</td>
<td>1.05</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>LAMPUNG</td>
<td>17.002</td>
<td>2.57</td>
<td>1.70</td>
<td>1.02</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>17.003</td>
<td>2.57</td>
<td>2.43</td>
<td>1.02</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>17.004</td>
<td>0.64</td>
<td>3.97</td>
<td>1.15</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>17.048</td>
<td>1.20</td>
<td>2.36</td>
<td>1.33</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>WEST JAVA</td>
<td>22.0462</td>
<td>0.72</td>
<td>2.81</td>
<td>1.08</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>22.07</td>
<td>1.26</td>
<td>2.37</td>
<td>1.04</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>22.073</td>
<td>1.88</td>
<td>1.14</td>
<td>1.22</td>
<td></td>
</tr>
</tbody>
</table>

Table 11 Relationship of IRI, ADT and Population Growth Before & After Project

<table>
<thead>
<tr>
<th>No</th>
<th>PROVINCE</th>
<th>LINK No:</th>
<th>TIME</th>
<th>ADT</th>
<th>IRI</th>
<th>GROWTH (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>DISTRICT</td>
</tr>
<tr>
<td>1</td>
<td>BENGKULU</td>
<td>130.081</td>
<td>Before</td>
<td>819</td>
<td>11.80</td>
<td>2.11</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>797</td>
<td>4.70</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>130.082</td>
<td>Before</td>
<td>819</td>
<td>17.00</td>
<td>2.11</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>797</td>
<td>4.80</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>130.061</td>
<td>Before</td>
<td>1123</td>
<td>9.20</td>
<td>0.56</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>1771</td>
<td>4.50</td>
<td>2.56</td>
</tr>
<tr>
<td></td>
<td></td>
<td>130.062</td>
<td>Before</td>
<td>1123</td>
<td>10.50</td>
<td>1.57</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>1771</td>
<td>4.60</td>
<td>3.04</td>
</tr>
<tr>
<td>2</td>
<td>SOUTH</td>
<td>15.016</td>
<td>Before</td>
<td>851</td>
<td>11.40</td>
<td>1.57</td>
</tr>
<tr>
<td></td>
<td>SUMATERA</td>
<td></td>
<td>After</td>
<td>742</td>
<td>5.90</td>
<td>3.04</td>
</tr>
<tr>
<td></td>
<td></td>
<td>150.211</td>
<td>Before</td>
<td>1651</td>
<td>11.20</td>
<td>1.57</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>3122</td>
<td>3.80</td>
<td>3.04</td>
</tr>
<tr>
<td></td>
<td></td>
<td>150.212</td>
<td>Before</td>
<td>1651</td>
<td>9.80</td>
<td>2.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>3122</td>
<td>4.30</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15.023</td>
<td>Before</td>
<td>1146</td>
<td>8.20</td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>757</td>
<td>4.70</td>
<td>1.45</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15.058</td>
<td>Before</td>
<td>3920</td>
<td>8.30</td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>4434</td>
<td>3.10</td>
<td>1.45</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15.059</td>
<td>Before</td>
<td>2178</td>
<td>10.40</td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>1465</td>
<td>4.70</td>
<td>1.45</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15.06</td>
<td>Before</td>
<td>1720</td>
<td>9.40</td>
<td>2.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>1352</td>
<td>7.80</td>
<td>9.67</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15.061</td>
<td>Before</td>
<td>1282</td>
<td>12.10</td>
<td>2.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>978</td>
<td>5.10</td>
<td>9.67</td>
</tr>
<tr>
<td>3</td>
<td>LAMPUNG</td>
<td>17.002</td>
<td>Before</td>
<td>5857</td>
<td>5.60</td>
<td>1.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>15030</td>
<td>3.30</td>
<td>1.76</td>
</tr>
<tr>
<td></td>
<td></td>
<td>17.003</td>
<td>Before</td>
<td>5857</td>
<td>11.20</td>
<td>1.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>15030</td>
<td>4.60</td>
<td>1.76</td>
</tr>
<tr>
<td></td>
<td></td>
<td>17.004</td>
<td>Before</td>
<td>4754</td>
<td>12.70</td>
<td>1.19</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>3037</td>
<td>3.20</td>
<td>0.54</td>
</tr>
<tr>
<td></td>
<td></td>
<td>17.048</td>
<td>Before</td>
<td>842</td>
<td>11.10</td>
<td>1.19</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>1006</td>
<td>4.70</td>
<td>0.54</td>
</tr>
<tr>
<td>4</td>
<td>WEST JAVA</td>
<td>22.0462</td>
<td>Before</td>
<td>1006</td>
<td>12.10</td>
<td>3.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>722</td>
<td>4.30</td>
<td>1.05</td>
</tr>
<tr>
<td></td>
<td></td>
<td>22.070</td>
<td>Before</td>
<td>1678</td>
<td>12.30</td>
<td>0.73</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>2117</td>
<td>5.20</td>
<td>1.38</td>
</tr>
<tr>
<td></td>
<td></td>
<td>22.073</td>
<td>Before</td>
<td>2956</td>
<td>4.10</td>
<td>0.84</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>5572</td>
<td>3.60</td>
<td>1.28</td>
</tr>
</tbody>
</table>

Source: Survey, 2004
Table 12 Relationship of IRI, ADT, In-migration and Outmigration Before & After Project

<table>
<thead>
<tr>
<th>No</th>
<th>PROVINCE</th>
<th>LINK No.</th>
<th>TIME</th>
<th>ADT</th>
<th>IRI</th>
<th>IN-MIGRATION (%)</th>
<th>OUT-MIGRATION (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>DISTRICT</td>
<td>SUB-DIST</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>DISTRICT</td>
<td>SUB-DIST</td>
</tr>
<tr>
<td>2</td>
<td>SOUTH</td>
<td>15,016</td>
<td>Before</td>
<td>851</td>
<td>11,40</td>
<td>6,51</td>
<td>2,5</td>
</tr>
<tr>
<td></td>
<td>SUMATERA</td>
<td></td>
<td>After</td>
<td>742</td>
<td>5,90</td>
<td>0,29</td>
<td>6,09</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Before</td>
<td>1651</td>
<td>11,20</td>
<td>6,51</td>
<td>2,5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>3122</td>
<td>3,80</td>
<td>0,29</td>
<td>6,09</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Before</td>
<td>1651</td>
<td>9,80</td>
<td>6,51</td>
<td>2,5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>3122</td>
<td>4,30</td>
<td>0,29</td>
<td>6,09</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Before</td>
<td>1146</td>
<td>8,20</td>
<td>1,67</td>
<td>10,17</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>757</td>
<td>4,70</td>
<td>2,07</td>
<td>1,56</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Before</td>
<td>3930</td>
<td>8,30</td>
<td>2,75</td>
<td>8,8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>4434</td>
<td>3,10</td>
<td>1,15</td>
<td>2,72</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Before</td>
<td>2178</td>
<td>10,40</td>
<td>2,75</td>
<td>8,8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>1465</td>
<td>4,70</td>
<td>1,15</td>
<td>2,72</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Before</td>
<td>1720</td>
<td>9,40</td>
<td>0,98</td>
<td>4,44</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>1352</td>
<td>7,80</td>
<td>2,03</td>
<td>1,7</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Before</td>
<td>1282</td>
<td>12,10</td>
<td>0,98</td>
<td>4,44</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>978</td>
<td>5,40</td>
<td>2,03</td>
<td>1,7</td>
</tr>
<tr>
<td>4</td>
<td>WEST JAVA</td>
<td>22,0462</td>
<td>Before</td>
<td>1006</td>
<td>12,10</td>
<td>28,64</td>
<td>12,22</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>722</td>
<td>4,30</td>
<td>10,69</td>
<td>2,19</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Before</td>
<td>1678</td>
<td>12,30</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>2117</td>
<td>5,20</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Before</td>
<td>2956</td>
<td>4,10</td>
<td>-3,1</td>
<td>-4,95</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>After</td>
<td>5572</td>
<td>3,60</td>
<td>0,67</td>
<td>23,2</td>
</tr>
</tbody>
</table>

2. Changes on migration
The increase of IRI cause by this rehabilitation project will push the increasing of ADT at roads because level of accessibility was increased. Relationship between road condition increasing and acceleration of migration can see at Table 10 & Table 11. Through that table, we can see some of roads especially for roads which connecting isolated region, immigration and outmigration number tend to increase in sharp skew, while for inter urban and near urban area the migration number tend to decrease. It may understood cause for region around city, the increasing of road condition make they moving coomuter without permanent migration, while for remote area the increasing of road condition push people to move in or out from they place.

3. Social statues changes
Survey result to the resident citizen around the roads which has rehabilitated, they said get double benefit, that is accessibility of transportation and economic advantages by the increasing of that road condition. Economically, people income increase 170% - 250% comparing to the condition before rehabilitation. See table 13. Off course the increasing of their income will influence they daily life.

C. Economic changes benefit
1. Changes on GRDP growth acceleration
When that road rehabilitation has done, directly will influence to the GRDP of construction sector. After this project has done then utilized with expectation the numbers of ADT will increase, it means the activity will increase too so GRDP of transportation sector will improve, more over make chain effect to others sector which relate with road uses will increase. Thus the road rehabilitation project will give big effect to GRDP. Beside GRDP, road rehabilitation project also influence to PAD (Regional Revenue) especially obtain through tax and retribution from any activity around the roads.

2. Changes on level of load and unload handling
According to the increasing of IRI and ADT, we expect the level of load and unload goods at port will improve, especially for commodities which will inter insuler and uses that roads as backbone for their tranportation. This relationship can see at table 15 below:

3. Changes on Land Price
One of direct benefit of the road rehabilitation which people feels is the changing on land price around that road. This increasing in associated with accessibility level can give by better road condition. Base on survey which has done, the increasing of land price is significant, that is increase 260% - 460 % average if comparing with land price before this rehabilitation project, even at South Sumatera the increasing of land price is very fantastic, that is 11 times. Off course the land price increase is not just influent by road rehabilitation as like social effect of increasing income. The increasing of land price off course will push the changes on activity pattern of the residence citizen and it means push the change of land use pattern.
Table 13 Road Rehabilitation Influence to Social & Economic Changes

<table>
<thead>
<tr>
<th>Province</th>
<th>Package No.</th>
<th>Link No.</th>
<th>From ... To ...</th>
<th>Length Km</th>
<th>Avg of ADT Increase</th>
<th>Avg of IRI Increase</th>
<th>Avg of Income Increase</th>
<th>Avg of Land price Increase</th>
<th>Avg of Travel Time Decrease</th>
<th>Avg of Accessibility Increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bengkulu</td>
<td>G-22</td>
<td>13.001</td>
<td>Kembang - Kepahyang</td>
<td>43.400</td>
<td>0.95</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>G-23</td>
<td>13.002</td>
<td>Kepahyang-Curup</td>
<td>24.000</td>
<td>0.69</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>South</td>
<td>N-13</td>
<td>13.008/1</td>
<td>Curup - Km 127</td>
<td>42.000</td>
<td>4.14</td>
<td>1.84</td>
<td>2.94</td>
<td>2.88</td>
<td>0.47</td>
<td>1.71</td>
</tr>
<tr>
<td></td>
<td>N-14</td>
<td>13.006/1</td>
<td>Km 127 - Mt.Aman/T.Sawah</td>
<td>45.000</td>
<td>2.47</td>
<td>2.94</td>
<td>2.88</td>
<td>2.88</td>
<td>0.47</td>
<td>1.71</td>
</tr>
<tr>
<td></td>
<td>G-30</td>
<td>13.006/2</td>
<td>Manna - Tanjung Kemuning</td>
<td>41.300</td>
<td>1.59</td>
<td>1.63</td>
<td>1.67</td>
<td>1.99</td>
<td>0.45</td>
<td>1.36</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Tanjung Kemuning - Linau</td>
<td>45.700</td>
<td>1.82</td>
<td>1.80</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sub Total</td>
<td></td>
<td></td>
<td></td>
<td>241.400</td>
<td>1.70</td>
<td>1.56</td>
<td>2.52</td>
<td>2.58</td>
<td>0.46</td>
<td>1.60</td>
</tr>
<tr>
<td>South</td>
<td>G-26</td>
<td>15.021/1</td>
<td>Kayu agung - Km 105 PLB</td>
<td>25.600</td>
<td>2.44</td>
<td>2.12</td>
<td>2.22</td>
<td>1.92</td>
<td>0.55</td>
<td>1.55</td>
</tr>
<tr>
<td>Sonatera</td>
<td>N-15</td>
<td>15.016</td>
<td>Palembang - Kayuagung</td>
<td>64.000</td>
<td>1.25</td>
<td>1.59</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>N-16</td>
<td>15.021/2</td>
<td>Km 105 PLB - Rasuan</td>
<td>50.450</td>
<td>0.50</td>
<td>1.70</td>
<td>2.22</td>
<td>1.92</td>
<td>0.55</td>
<td>1.55</td>
</tr>
<tr>
<td></td>
<td>G-30</td>
<td>15.058</td>
<td>Pangkal Pinang - Namang</td>
<td>20.600</td>
<td>4.90</td>
<td>1.24</td>
<td>1.90</td>
<td>21.42</td>
<td>0.50</td>
<td>2.00</td>
</tr>
<tr>
<td></td>
<td>G-31</td>
<td>15.059</td>
<td>Namang - Koba</td>
<td>35.400</td>
<td>7.03</td>
<td>1.55</td>
<td>1.90</td>
<td>21.42</td>
<td>0.50</td>
<td>2.00</td>
</tr>
<tr>
<td></td>
<td>N-22</td>
<td>15.06</td>
<td>Koba - Airbara</td>
<td>9.000</td>
<td>3.03</td>
<td>1.76</td>
<td>2.06</td>
<td>13.97</td>
<td>0.50</td>
<td>1.56</td>
</tr>
<tr>
<td></td>
<td>15.061</td>
<td>Airbara - Tuoali</td>
<td>57.000</td>
<td>2.81</td>
<td>2.27</td>
<td>2.06</td>
<td>13.97</td>
<td>0.50</td>
<td>1.56</td>
<td></td>
</tr>
<tr>
<td>Sub Total</td>
<td></td>
<td></td>
<td></td>
<td>284.300</td>
<td>2.00</td>
<td>1.72</td>
<td>2.13</td>
<td>11.03</td>
<td>0.51</td>
<td>1.72</td>
</tr>
<tr>
<td>Lampung</td>
<td>G-31</td>
<td>17.002</td>
<td>Teggineng - Gunung Sugih</td>
<td>25.400</td>
<td>3.61</td>
<td>1.32</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>G-32</td>
<td>17.004</td>
<td>Terbanggi Besar - Kota Bumi</td>
<td>37.400</td>
<td>3.35</td>
<td>1.24</td>
<td>1.41</td>
<td>3.05</td>
<td>0.51</td>
<td>1.64</td>
</tr>
<tr>
<td></td>
<td>N-23</td>
<td>17.048</td>
<td>Ketapang - Gunung Labuhan</td>
<td>44.600</td>
<td>1.42</td>
<td>1.42</td>
<td>2.00</td>
<td>3.46</td>
<td>0.55</td>
<td>1.53</td>
</tr>
<tr>
<td>Sub Total</td>
<td></td>
<td></td>
<td></td>
<td>119.200</td>
<td>3.67</td>
<td>1.43</td>
<td>1.71</td>
<td>3.26</td>
<td>0.53</td>
<td>1.59</td>
</tr>
<tr>
<td>West Java</td>
<td>G-39</td>
<td>22.073</td>
<td>Indramayu - Karangampel</td>
<td>27.000</td>
<td>5.02</td>
<td>0.62</td>
<td>2.54</td>
<td>2.62</td>
<td>0.48</td>
<td>1.60</td>
</tr>
<tr>
<td></td>
<td>22.07</td>
<td>Jatibarang</td>
<td>42.000</td>
<td>6.01</td>
<td>1.86</td>
<td>1.90</td>
<td>6.67</td>
<td>0.48</td>
<td>1.55</td>
<td></td>
</tr>
<tr>
<td></td>
<td>N-28</td>
<td>22.046/2</td>
<td>Sukamakara - Sindangbarang</td>
<td>66.300</td>
<td>3.08</td>
<td>1.63</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sub Total</td>
<td></td>
<td></td>
<td></td>
<td>129.300</td>
<td>4.70</td>
<td>1.37</td>
<td>2.22</td>
<td>4.64</td>
<td>0.48</td>
<td>1.70</td>
</tr>
</tbody>
</table>

Source: IRMS & Survey 1998

Table 14 Increasing of ADT ,IRI and GRDP

<table>
<thead>
<tr>
<th>No:</th>
<th>PROVINCE</th>
<th>LINK NO</th>
<th>INCREASE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>ADT</td>
</tr>
<tr>
<td>1</td>
<td>BENGKULU</td>
<td>13.0081</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td></td>
<td>13.0082</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td></td>
<td>13.0061</td>
<td>1.58</td>
</tr>
<tr>
<td></td>
<td></td>
<td>13.0062</td>
<td>1.58</td>
</tr>
<tr>
<td>2</td>
<td>SOUTH</td>
<td>15.016</td>
<td>0.87</td>
</tr>
<tr>
<td></td>
<td>SUMATERA</td>
<td>15.0211</td>
<td>1.89</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15.0212</td>
<td>1.89</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15.023</td>
<td>0.66</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15.058</td>
<td>1.13</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15.059</td>
<td>0.67</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15.06</td>
<td>0.79</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15.061</td>
<td>0.76</td>
</tr>
<tr>
<td>3</td>
<td>LAMPUNG</td>
<td>17.002</td>
<td>2.57</td>
</tr>
<tr>
<td></td>
<td></td>
<td>17.003</td>
<td>2.57</td>
</tr>
<tr>
<td></td>
<td></td>
<td>17.004</td>
<td>0.64</td>
</tr>
<tr>
<td></td>
<td></td>
<td>17.048</td>
<td>1.20</td>
</tr>
<tr>
<td>4</td>
<td>WEST JAVA</td>
<td>22.0462</td>
<td>0.72</td>
</tr>
<tr>
<td></td>
<td></td>
<td>22.07</td>
<td>1.26</td>
</tr>
<tr>
<td></td>
<td></td>
<td>22.073</td>
<td>1.88</td>
</tr>
</tbody>
</table>

Table 15 ADT, IRI & loading & unloading

<table>
<thead>
<tr>
<th>NO</th>
<th>REGION</th>
<th>ROAD CONDITION (%)</th>
<th>LOAD &amp; UNLOAD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>BEFORE PROJECT</td>
<td>AFTER PROJECT</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ADT</td>
<td>IRI</td>
</tr>
<tr>
<td>1</td>
<td>Bengkulu</td>
<td>46.58</td>
<td>150.75</td>
</tr>
<tr>
<td>2</td>
<td>South Sumatera</td>
<td>222.37</td>
<td>115.71</td>
</tr>
<tr>
<td>3</td>
<td>Lampung</td>
<td>98.86</td>
<td>165.05</td>
</tr>
<tr>
<td>4</td>
<td>West Java</td>
<td>150.36</td>
<td>135</td>
</tr>
</tbody>
</table>

297
D. Benefit on Physical and Environment Changes

1. Changes on land use
Increasing of road condition indirectly will push changes on activity pattern. The change of activity pattern will push changes on land use and spatial as like the shift of agriculture land to resident, industrial or another non agriculture land uses. Data obtain show the shift of land use more faster associate with increasing of roads condition, it is clearly on acceleration number of land use changes.

2. Changes on the wide of production land and production level for each commodity
In relation with changes on land use pattern, data obtain for Province Bengkulu, especially for Kabupaten Rejang Lebong show statistically the wide of padi harvest tend to decrease after road rehabilitation. This decreasing arround 15-25% comparing with condition before rehabilitation, as well production numbers also decrease between 12-14%. For South Sumatera, especially for Kabupaten Komering Ulu (OKU) the wide farm crops and production numbers decrease 8 – 11%, and this decreasing continued until 1997.
But for Kabupaten Bangka and Pangkal Pinang, the occres is positive influence, whereas the wide of padi harvest and production numbers of wetland padi and dryland padi increase 60-120% average, even for kecamatan Pangkalan Baru increase till 300% or 3 times to condition before road rehabilitation.
For Lampung, especially at Kabupaten Lampung Utara and Tengah the wide of wetland padi harvest increase 13-30 % average, and production numbers increase till 10 -17 %. While dryland padi decrease till 14-15% and production numbers decrease 70% comparing to condition before road rehabilitation. At Kabupaten Lampung Selatan wide wetland padi increase 5,5 times and production numbers increase 11 times, in other side dryland padi decrease 84 - 90 % if comparing with condition before road rehabilitation.
At west Java, especially at Kecamatan Sukananagara and Tanggeung, Kabupaten Cianjur, the wide of wetland padi harvest increase arroun 11%, and production numbers increase till 17 %. While dryland padi decrease till 25% and production numbers decrease 19% if comparing between after and before road rehabilitation. The relationship of ADT, IRI and production level of main commodities show at Table 16 below.

Table 16 ADT, IRI & Production Level of Main Commodities

<table>
<thead>
<tr>
<th>No</th>
<th>REGION</th>
<th>ROAD CONDITION (%)</th>
<th>MAIN COMMODITIES</th>
<th>DISTRICT</th>
<th>SUB DISTRICT</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Bengkulu</td>
<td>46,58</td>
<td>Padi Sawah</td>
<td>(12-14)</td>
<td>Padi sawah</td>
</tr>
<tr>
<td>2</td>
<td>Sumatera Selatan</td>
<td>222,37</td>
<td>Palawija</td>
<td>(8-11)</td>
<td>Padi sawah</td>
</tr>
<tr>
<td>3</td>
<td>Lampung</td>
<td>98.86</td>
<td>Padi sawah</td>
<td>10-17</td>
<td>Padi Ladang</td>
</tr>
<tr>
<td>4</td>
<td>Jawa Barat</td>
<td>150,36</td>
<td>Padi sawah</td>
<td>17</td>
<td>Padi Ladang</td>
</tr>
</tbody>
</table>

3. Changes on accessibility level
One of expect benefit from road rehabilitation is good and stable road length added. In fact it occurs, for examples for West Java especially at Kabupaten Majalengka the road legth with good and stable condition increase from 76 Km in 1990 - 1993 period become 756 Km at 1997. The added of good road length make the ratio between road length and wide region (as an indicator of accessibility) increase. At Majalengka this ratio increase from 0,00063 to 0,00628 Km/Km2. It show the level of accessibility for this region becomes better.
Peoples opinion also said that after road rehabilitation the level of accessibility increase 1,6 - 1,7 times comparing with condition before rehabilitation. They also said that travel time of road user increase 40 - 55 %

E. Benefit on Bridge condition increase
The main benefit of the bridge rehabilitation is obtain well maintain for level of service of the bridge, so it can services traffic increase in that road whereas the bridge are. In average after rehabilitation the capacity of the bridge increase 7-52%, it utilize to avoid bottle neck arroun the bridge which it can substruct road performance.

4. CONCLUSION AND RECOMENDATION

Road Rehabilitation Benefit
Base on the above discussion can take some conclusion in relation with road rehabilitation which has been done between 1990 – 1998 in Bengkulu, South Sumatera, Lampung and West Java provinces. Direct benefit is:
1. IRI changes with increase 140% in average than condition before rehabilitation, even for South Sumatera increase till 222%.
2. Changes on traffic volume (ADT), in relation with IRI of the road, ADT increase 140 % in average, even for Lampung and Bengkulu increase till 150% - 165%. But the fantastic increasing occures in West Java and Lampung whereas increase between 360%-470%.
3. Changes on travel speed, in relation with IRI increasing occurs increasing on travel speed so we get formula as below:

\[
\text{Speed} = -0.13443 \times \text{IRI} + 49.71993 \\
R = 0.98
\]
4. Changes on vehicle composition, very interesting is base on evaluation the numbers of sedan, passenger car and mini-bus which it ordinary private vehicle increased in numbers, but for bus and truck which it ordinary commercial vehicle tend to decrease, except in the links at South Sumatera, Lampung and Java. Meanwhile the road links in P. Bangka and Bengkulu only private car has increase. So private car get more benefit from this rehabilitation project, meanwhile commercial car only a little bit use this road improvement. May be it occurs cause by the condition of commercial vehicle is not yet suitable.

5. Change on Vehicle Operating Cost (VOC). The VOC decrease 21-46 % in average comparing with VOC before road rehabilitation

6. Change on Road Maintenance Cost. Better road after rehabilitation will substract maintenance cost and it an economic advantage for road operator. After rehabilitation maintenance cost reduce 35% in average.

7. BCR and EIRR of the roads. In average BCR of the road rehabilitation project is 4,38 with average benefit 30 milyar rupiahs each link if calculate since 1990 till 2010. This condition is very good to support transportation system improvement and regional economic activity improvement.

Beside direct benefit as it has mentioned, that road rehabilitation also gives indirect benefit, such as:

1. Social changes benefit: Changes on population growth acceleration, Changes on migration acceleration, Social statues changes etc.
2. Economic changes benefit: Changes on GRDP growth acceleration, Changes on production distribution for each commodity, Changes on level of load and unload goods handling at port, etc.
3. Physical and environmental benefit: Changes on land distribution (land use pattern), Changes on productive land Changes on accessibility

Road Rehabilitation Impact

In relation with road rehabilitation, beside it gives us benefit, we must wary about impact will rise, as like:

1. Level of traffic accident, needs to wary about this happen
2. Changes of wide productive land especially for food and crops, which at some region tend to decreasing of wide harvest land and production numbers of wetland padis and dry-land padis, also crops, especially around the road near urban area which many land change to industries, residence and other non agriculture use.
3. The increasing of accessibility will push migration level, if it is not following with economic distribution and development as like distribution of social economic facility, will push copious in-migration to urban area which it raise social problem
4. The following impact causes the increasing of land price will push acceleration on people activities pattern and it means acceleration on land use pattern changes.

5. REFERENCES


TDM Encyclopedia.(April 4, 2006). Measuring Transport, Victoria Transport Policy Institute,

TDM Encyclopedia.(December 14, 2005). Economic Development Impacts, Victoria Transport Policy Institute,

TDM Encyclopedia. (May 9, 2005). Accessibility, Victoria Transport Policy Institute,
A REVIEW ON INDONESIA’S HIGHWAY BRIDGE CONSTRUCTION SPECIFICATION IN ORDER TO SUPPORT TRANS ASIAN HIGHWAY

**PAPER TITLE**

(A 90 Characters Max)

**TRACK**

**AUTHOR**

(Capitalize Family Name)

**POSITION**

**ORGANIZATION**

**COUNTRY**

Rulli Ranastra Irawan

Researcher

IRE

Indonesia

**CO-AUTHOR(S)**

(Capitalize Family Name)

**POSITION**

**ORGANIZATION**

**COUNTRY**

**E-MAIL**

(for correspondence)

rulli.ranastra@pusjatan.pu.go.id

rulli.ranastra@gmail.com

**KEYWORDS:**

Bridge, Construction, Specification, Quality, Deterioration

**ABSTRACT:**

Bridge as part of infrastructures automatically have important contribution to the functionality of road networks. Moreover Indonesia consists of several island and province with specific site condition need bridge infrastructures. So, it is essential to make sure all aspect of bridge establishment is conform with standard. Based on BMS data in 2007, Indonesia has 88,900 (1060 km) bridge consists of 54,000 bridge in district road and 35,000 bridges in national and provincial road. In recent years, the construction of every new bridge funded by the government are guided by DGH’s general standard specification for road and bridge. Some case in Indonesia’s road network shows, newly built bridges already experience early deterioration due to during construction problems. In order to support the building of the trans asian highway, the quality of bridge construction has to be in the same level along the path of the trans asian highway. So it is very important to review the current specification.
A Review On Indonesia’s Highway Bridge Construction Specification In Order To Support Trans Asian Highway

Rulli Ranastra Irawan
Institute of Road Engineering, Bandung INDONESIA.
Email for correspondence: rulli.ranastra@pusjatan.pu.go.id

1. INTRODUCTION

Bridge as part of road networks generally control the functionality of road networks system. Moreover in Indonesia which have several island and river, made infrastructures such as bridges become an essential part of road networks. In Indonesia, road network consist of 88,900 bridges with total length of 1,060 km. It means 0.25% from total length of road networks. If we look further from those numbers, approximately 16,962 bridges (325 km) located on national road, 18,038 bridges (335 km) and the rest 50,000 bridges (400 km) on district and municipal road (Sub Dit.Jembatan 2013).

<table>
<thead>
<tr>
<th>Table 1 Distribution of bridge span in Indonesia</th>
<th>Percentage of distribution (nos)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-20</td>
<td>78%</td>
</tr>
<tr>
<td>20-30</td>
<td>9%</td>
</tr>
<tr>
<td>30-60</td>
<td>9%</td>
</tr>
<tr>
<td>60-100</td>
<td>2%</td>
</tr>
<tr>
<td>&gt; 100</td>
<td>2%</td>
</tr>
</tbody>
</table>

(Compiled from Sub Dit.Jembatan, 2013)

Based on the main span, the distribution of bridges in Indonesia can be seen on Table 1. It is clear that majority Indonesia have a short span bridge (0-20m) with percentage around 78%. Only 2% for the long span bridges, but running by the time the need for the construction of long span bridges increasing in line with the economy development. Also, there are several types of bridges in Indonesia (Table 2) such as culvert, girder, truss, etc. Error! Reference source not found. shows a bridge with girder for the upperstructure have a large population with percentage around 69%.

<table>
<thead>
<tr>
<th>Table 2 Bridge distribution in Indonesia based on type</th>
<th>Percentage of distribution (nos)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Culvert</td>
<td>17%</td>
</tr>
<tr>
<td>Girder</td>
<td>69%</td>
</tr>
<tr>
<td>Truss</td>
<td>10%</td>
</tr>
<tr>
<td>Other</td>
<td>5%</td>
</tr>
</tbody>
</table>

(Compiled from Sub Dit.Jembatan, 2013)

If we take a look to the bridge condition in Indonesia (Table 3), most of It were having small damage (Condition 1) with minor countermeasure to repair it. Three percent were failed because several factors such as overloading, natural disaster, etc.
Table 3 Bridge distribution in Indonesia based on condition

<table>
<thead>
<tr>
<th>Condition</th>
<th>Percentage of distribution (nos)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 (good)</td>
<td>16%</td>
</tr>
<tr>
<td>1 (minor damage)</td>
<td>22%</td>
</tr>
<tr>
<td>2 (moderate damage)</td>
<td>15%</td>
</tr>
<tr>
<td>3 (heavily damage)</td>
<td>8%</td>
</tr>
<tr>
<td>4 (critical)</td>
<td>6%</td>
</tr>
<tr>
<td>5 (failed)</td>
<td>3%</td>
</tr>
</tbody>
</table>

(Compiled from Sub Dit.Jembatan, 2013)

From the point of view of national importance, although the length of bridges is only 0.25% from the existing road length, but bridges hold strategic and very important role for the operational sustainability of the transportation network itself. That is because of the connectivity function of bridges, without it there will be no road network and the road segment become malfunction.

Moreover, based on its speciality, the construction of bridges will involving large number of initial investment compared to ordinary road. Efforts to anticipate the higher risks of failures including major defects which makes the lumps of transportation system and safety become a necessity. While road is infrastructure for land transportation and become major transportation in this country, then of course any damage to the bridge will be disrupted continuity of the economy of this country. Therefore the government's attention to the bridge which is generally planned to have a longer service life will become a real concern in the development of the country. So it is not just the physical construction of the bridge only, because the bridge will also affect the economic prosperity and pride of the people.

2. BRIDGE MANAGEMENT SYSTEM IN INDONESIA

Starting in 1990’s Directorate General of Highway, Ministry of Public Work has introduced Bridge Management System containing guidelines for bridge works starting from bridge planning guidelines to bridge rehabilitation guidelines, that can be used by bridge designers, owners, contractors, supervisors and rehabilitation applicators. The following are necessary component in a bridge works and guideline in the surrounding system.

In 1989, cooperation with Australia were carried out to develop comprehensive Design Code for Bridge. The cooperation were long enough, until in 1992 there were produced 17 modules, what we called it Bridge Manajemen System (BMS) see Table 4. Those modules are comprehensive, contains all bridge activities, from bridge operational management including design code, and that is not less interesting is the availability of the manual on how to use those codes. These how to use manual, became practical on the field to choose and specify the appropriate type of construction, which make preliminary design of bridge more easier. Because the substance and scope of the discussion is very broad, it make BMS greatly facilitate the designers in Indonesia to implement bridge construction design activities, especially for bridges with span up to 100 meter.
Table 4 Component and Available Guideline of Bridge Works

<table>
<thead>
<tr>
<th>Component</th>
<th>Guidelines</th>
</tr>
</thead>
<tbody>
<tr>
<td>All Inspection</td>
<td>IBMS General Procedure Guideline</td>
</tr>
<tr>
<td>Emergency Action</td>
<td>Maintenance &amp; Rehabilitation Guideline</td>
</tr>
<tr>
<td>IBMS Management Information System</td>
<td>IBMS Management Information System Guideline</td>
</tr>
<tr>
<td>Planning and Programming</td>
<td>IBMS Planning and Programming Guideline</td>
</tr>
<tr>
<td>Investigation and Technical Design</td>
<td>Bridge Investigation</td>
</tr>
<tr>
<td></td>
<td>Bridge Design Manual</td>
</tr>
<tr>
<td>Bidding Document</td>
<td>Standard Specification for Bridge Construction</td>
</tr>
<tr>
<td>Bidding Process</td>
<td>Standard Specification for Bridge Rehabilitation</td>
</tr>
<tr>
<td></td>
<td>Bridge Specifiers Guideline</td>
</tr>
<tr>
<td>Bridge Material Management</td>
<td>Bridge Unit Supply System</td>
</tr>
<tr>
<td></td>
<td>Storage Management Guideline</td>
</tr>
<tr>
<td>Construction</td>
<td>Inventory System Guideline</td>
</tr>
<tr>
<td>Maintenance &amp; Rehabilitation</td>
<td>Supervision Guideline</td>
</tr>
<tr>
<td>Monitoring</td>
<td>Construction Guideline</td>
</tr>
<tr>
<td></td>
<td>Maintenance &amp; Rehabilitation Guideline</td>
</tr>
<tr>
<td></td>
<td>General Procedures/ IBMS – MIS Guideline</td>
</tr>
</tbody>
</table>

In the autonomous era in which Local Government has important roles in infrastructures become increase, many road and bridge built as an effort to develop a region. As a results, there is a possible lack of control management and without proper management this will end with disaster, as deterioration may occur (see figure 1), which maintenance and rehabilitation is necessary to conduct properly.

Figure 1 Bridge general deterioration model
3. TRANS ASIAN HIGHWAY IN INDONESIA

Trans Asian Road Network in Indonesia will have two routes that cross the territory of Indonesia namely Route (AH2) or (A2) includes the western part of the island of Bali to connect all major cities to the northern region of the island of Java to Singapore and Malaysia to continue up to Khosravi , Iran and route (AH 25) or (A 25) ranging from Banda Aceh covers the entire East side of the main towns on the island of Sumatra, and then met with the route (AH 2) or (A 2) throughout the entire territory of the southern part of Java Island continued up to the island of Bali.

![Asian Highway Routes in Indonesia](http://fr.wikipedia.org/wiki/Reseau_routier_asiatique)

The two routes which are AH2, 13,177 km (8326 miles); Denpasar, Indonesia to Khosravi, Iran and AH25, 2,549 km (1593 miles); Banda Aceh, Indonesia to Merak, Indonesia (on AH2) as seen in figure 2 are very strategic. More than 100 bridges constructed on the routes. When among those bridges loosing their serviceability, paralysis of traffic on the route is inevitable.

4. CURRENT SPECIFICATION

Highway construction industry services, when compared to other construction services industries such as manufacturing, for example, is relatively very conservative. Although a lot of road construction technology has developed an innovative, system procurement of road infrastructure is relatively stagnant with DBB approach (design-bid-build).

In DBB approach, the design carried out separately from the construction. DBB process involves detailed planning starting from the procurement of consultancy services for the design, implementation design, preparation of specifications, volume and cost analysis, construction procurement through open announcement to the determination of the winner is based on the lowest price that can be accounted for (the responsiveness and lowest responsible bidder). Everything form the implementation requirements, such as Environmental Impact Analysis (EIA/AMDAL), land acquisition, coordination with authorities river, rail, and public utilities are the responsibility of the service user and need to be finalized before the auction begins. This is done to limit the scope of work the contractor establish a clear and eliminate uncertainty during execution. Quality of execution is controlled by the application of prescription specifications (prescriptive) combined with inspection and acceptance procedures based on the principles of statistics through a representative or supervisory activities in the field.

The current specification (figure 3) for bridge construction in Indonesia are developed by Directorate General of Highway which consists of 10 (ten) division that arrange everything to do and not to do by the three stake holders in construction works of roads and bridges. The three stake holders are the owner, the contractor and the supervision consultant.
Adopted contract system in this specification are unit price contract for all major works, only for several minor work activities uses lumpsum payment.

- Division 1 - General
- Division 2 - Drainage
- Division 3 - Ground Work
- Division 4 - Pavement Widening And Road Shoulder
- Division 5 - Grained Pavement
- Division 6 - Asphalt Pavement
- Division 7 - Structure
- Division 8 - Conditions Restoration And Minor Work
- Division 9 - Daily Jobs
- Division 10 - Routine Maintenance Work

In this paper will specifically discuss the division 7 - structure which is a major division in bridge work.

- 7.1 Concrete
- 7.2 Prestressed Concrete
- 7.3 Steel Reinforcement
- 7.4 Steel Structures
- 7.5 Erection Of Steel Bridge
- 7.6 Pile Foundation
- 7.7 Caisson Foundation
- 7.8 Mortar
- 7.9 Stone Masonry
- 7:10 Riprap and Gabions
- 7:11 Expansion Joints
- 7:12 Bearing
- 7:13 Railing
- 7:14 Bridge Identification Plate
- 7:15 Disassembly of Structure
- 7:16 Bridge Deck Drainage

Which in almost all those section contain these following sub-section.

- General
- Materials
- Equipment
- Implementation
- Quality Control, and
- Measurement and Payment

Although this DBB provide fair rules of the game for service providers, this approach has many limitations, such as:

- This approach requires a relatively long time provision.
- Procurement with the principle of the lowest prices often provide a narrow space to get the quality and timeliness of products that are critical to the long-term performance.
- Specifications of rigid prescription with no incentive to support the life cycle cost approach for a road project.
- Risks relating to the relationship between the technology used by the post-construction performance is not the responsibility of the contractor, since specifications determined by the owner.
• Specifications prescription and lowest bid prices do not provide incentives for contractor innovation. Innovation will lead to the complexity of the contract changes that are sensitive to abuse and ultimately difficult to implement.
• Contractor’s input in design, construction methods and quality specifications are relatively small and often end up in polemic disputes or the auditor's findings.
• The system requires owner’s staff in large quantities for the management of the design process, preparatory construction, construction period, and post-construction. Including the conduct preliminary design, even the design revision process design reviews, resolve problems in Environmental Impact Analysis, land acquisition, project management, quality control and maintenance. It is becoming increasingly difficult in recent times due to the limited recruitment of employees as a result of a paradigm shift in the role of government from rowing becoming steering.
• No incentives / disincentives for work disruption in traffic flow. Road preservation works, such as reconstruction, resurfacing, rehabilitation and routine maintenance requiring high quality products and fast processing time to avoid traffic disruption during construction.
• With the limited budget and the implementation of the road rather than the needs of the increasing demands of the user community, the alternative procurement systems becomes an urgent need. Developing alternative procurement system should be directed to overcome some disadvantages of traditional contracts, especially in an effort to achieve the life-cycle cost and lower risk well managed. This can be done through a variety of schemes, among others:
  o Reduced construction time through incentives / disincentives implementation period. In some road sections with high-traffic, construction disruption in traffic will cause tremendous congestion which risk must be borne by the service provider.
  o Variations of another scheme that can be used is by controlling the time of interruption of work on traffic flow
  o Reduce the risk of post-construction, particularly with regard the defect and deficiencies that are not captured in the process of inspection and acceptance of products work at the construction stage.
  o Reduce the uncertainty of cost and construction time. Number of unexpected things that are unanticipated in the design phase of cause many construction projects have later additional costs (or reduction of effective targets) and the addition of time (or require acceleration of workmanship which often then compromising the achievement of quality).
  o Reduce overall procurement time supply process. This can be done with a procurement process that is integrated.
5. PROPOSED SPECIFICATIONS

Various forms of innovative contract is essentially based on a proper setting of the degree of integration of the processes and the application of the provision of the type specifications in accordance with the level of integration is used. Various forms of innovative procurement, among others:

- The traditional contracts (DBB) modifications, such as:
  - DBB on guarantee
  - DBB with multi-parameter, eg:
    - Incentives costs + time. It is commonly practiced in PT Jasa Marga example.
    - Incentives costs + time + quality. For example, if the degree of compaction obtained better, then the payment is raised. This has never been applied in Indonesia
    - Application of lane rental for incentives / disincentives traffic disruption during construction.

- Form of Design and Build Contract (DB), several variants of the DB, among others:
  - DB with assurance
  - DB + operation + maintenance

- Form of Design-Build-Operation-Maintenance Contract. This is often referred to as the Performance Based Contract.

Each form of the contract requires the correct specification and generally can be divided into 3 kinds of specifications (Figure 4), namely:

- Input Based Specification or Prescriptive Specification.
  This specification is based on the determination of methods and materials to be used by the contractor.
- Output Based Specification.
These specifications define the quality of a product based on its relationship with performance (Performance Related Specification). These specifications define the desired product quality based on a certain value karasteristik product that has a correlation with the desired performance. Variations of this specification allows for the application of the payment adjustment based on the quality that is associated with effects on life-cycle cost.

- Specification based on performance (Performance-Outcome-Based Specification).
  
  This specification establishes the performance of the product during the term of the contract that are integrated.

![Figure 4 Specification and integration process of asset management](image)

Certain forms of contracts generally require a combination of these specifications with different emphasis. Design-Build form of contract-Maintain example would be dominated by the specification based on performance (Performance-Outcome-Based Specification) but can have a prescriptive elements and outputs for a particular specialized job if needed.

5.1 PERFORMANCE BASED CONTRACT (PBC)

PBC is a type of contract that bases payments on meeting minimum performance indicators. PBC has several potential advantages over traditional approaches, such as:

- Cost savings in road asset management and maintenance.
- Contractor has room for innovation in a competitive and responsible.
- Certainty of funding needs and financing certainty panjang.Sifat term contracts are multi-year and definitely a calculated risk. The risks of contracting decisions is the responsibility of the contractor.

Successful implementation of PBC requires some requirements, essentially, the application of PBC requires a shift in work culture, both as an organizer of the service users, service providers, and society as a road user. PBC does not reduce the responsibility of the organizers of the way, but the responsibility of the organizers to change the focus of a radical way. In PBC, the organizers do not need to set detail how the contractor to achieve the desired results.

The organizers will be required to be able to clearly define the problem, develop a methodology of determining acceptable performance indicators and measurable according to the organizers of the mission, and to develop objective performance evaluation system. Determination of performance indicators not only requires engineering expertise micro multi-field, but also to be able to solve the realistic achievement of macro indicators such as implied in the mission of the organizers.

PBC requires a culture shift service providers. Technical capabilities and innovation service providers needed to be competitive. Internal awareness and responsibility for the quality becomes inevitable to reduce the cost of construction contractors and increase profits. Pattern construction services business will also change with the increasing integration of the design, construction, operation and maintenance. PBC also requires a culture change in service users, given that most risks can occur due to the behavior of service users. Many of the
assumptions used in the design of roads and bridges is maintained through a variety of traffic regulations on road transport. One important example is the traffic loading.

Disobedience to the rules of the road user charge heaviest axis (MST) will cause uncertainty in the design strength of the structure. As a result, the reliability of the design will decrease which leads to premature failure risk. This risk is borne by the contractor if will cause a sizeable premium and a burden on the budget. This will complicate the handling strategy and budgeting, given the uncontrolled breach of the rules has a wide space of uncertainty due to the loss of control boundaries.

The things that need to be prepared in terms of service users for the successful implementation of PBC, among others, changes in the various forms of regulation / legislation to accommodate various forms of contract and specification of non-traditional, such as changes to the rules of government procurement of goods and services and their derivatives. Providing settings that are governed by Presidential Decree 70/2012, although it has to accommodate a variety bento integrated contract, but still adhering to the achievement of product by volume (input-output-based), not the starting point towards the creation of space innovation service providers as well as performance-based lump sum contract.

5.2 APPLICATION OF EXTENDED WARRANTY PERIOD

It is recognized that the application of the PBC requires thorough preparation and controlled. Wise strategy to be applied is a gradual manner while establishing an environment conducive to the implementation of PBC in full, for example:

- Application of the extended Warranty (Extended Warranty Period) in the early stages to build an internal quality culture of contractors.
- Application of the DB contract and variations to build innovation capability and the service provider the ability of service users in the management of an integrated contract.
- Implementation of PBC, with the scheme of Design-Build-Maintain-Warranty and variations in its various forms in full when service users, service providers and road users having sufficient readiness.

Application of EWP aimed at creating awareness of internal quality of service providers, in particular contractor. In the scheme of EWP contracts, contractors are required to develop a culture of quality implementation internally. This is done by transferring the risk of not achieving the quality of the implementation of the contractor for a period longer than the usual. In the event of a failure at this extended maintenance period, the contractor is responsible for repair or replacement.

Maintenance period is known in other terms with similar meaning roughly, among others Defect Notification Period, Defect Liability Period, Maintenance Warranty Period, Maintenance Period, and Defect Correction period. The length of the maintenance period by reference and existing regulations stipulated as follows:

- According to (FIDIC , 2008) section 1.1.3.7, the maintenance period is 12 months or as otherwise provided in the contract.
- According to the Presidential Decree 54/2010 and Perpres70 / 2012 which is an amendment of Presidential Decree 80/2003 states: minimum maintenance period of 6 months for a permanent job retention, and a minimum of 3 months for semi-permanent. He also explained that the maintenance period may exceed the budget.

Can be concluded that the maintenance for more than one year are not banned and here are no restrictions on how many years of maintenance beyond the budget year. Maintenance period is commonly applied in the physical contracts are 6 months (permanent physical state budget) to 1 year (permanent physical loan). Although there is no limit on the length of the maintenance, both in terms of the legislation in force, nor of reference given by FIDIC, the extension of the selected maintenance period is 2 years. Simply put, if a malfunction occurs, then it will be the responsibility of the contractor if the examination results proved that the damage caused by the failure of the contractor to meet the specs This is despite the passage of contractor of inspection and quality control performed by User Service / Supervision Consultant. Inspection and quality control systems are based on approaches or statistical representations that have a certain level of reliability limits. The scheme guarantees against defects
and deficiencies in the maintenance of the point is to fill the weaknesses method of inspection and quality control during construction.

5.3 APPLICATION ON PILOT PROJECTS

Application of EWP in Fiscal Year of 2008 implemented as part of the learning and applied to the road sections selected by the following criteria:

- It is a multi-year project (multi-years contract). This is done to provide economies scale sufficient for a contractor to build the capabilities achievement of quality.
- Not be a periodic and routine maintenance projects. This is to avoid uncertainty remaining life of the structure.
- Not on the location of the historically problematic. These criteria are to avoid factors risks due to design problems.
- Traffic Load relatively measurable and predictable (predictable).
- Has the number of service providers and adequate technical competence.

In FY 2008, there are 2 bridges work packages that implement EWP scheme as a learning project, namely:

1. Bridge Construction in West Coast South Sulawesi and
2. Cut Mutia Flyover Construction

6. CONCLUSION AND DISCUSSION

The growing challenge of road management in facing of increasingly high demands of society and the imbalance availability of budget with demanding needs innovation in road management, especially in terms of road asset management method and delivery system as an integrated system. Asset management needs to be directed towards meeting the minimum life-cycle-cost with priority based on the risk profile of each road segment. Risk of performance failure during this which entirely on the government as the organizer must be managed towards a fair risk sharing and proportional to the parties of service providers. The way towards healthy management of risk sharing among others, by creating a form of contract that is able to hold the interest of the owner to the service provider. Form of performance-based contracts (PBC) is a comprehensive integrated form of contract and able to provide space to service providers for innovation and risk-sharing more equitable and proportionate. The application form of PBC contracts and other innovative contracts requiring work culture changes both in the owner side, service providers and road users. The turn to the implementation of an integrated form of contract needs to be managed controllable and gradually. EWP form of contract is one of the initial stages of the implementation of PBC with the goal of building an internal quality culture among service providers. This internal quality culture is an important prerequisite for all stake holders to the successful implementation bridges management according to its life cycle cost based on the design.

REFERENCES

Nanotechnology for Green Roads

Dr Ajay Ranka¹, Dr Prakash Mehta² and Vivek Kane³

Zydex Industries, Vadodara, Gujarat, India
Email for correspondence: ashilpathak@zydexindustries.com

1 GREEN ROADS

We have taken this earth on lease from our future generations. It is our duty to return it to them in liveable condition if not better. Striving for sustainability through prudent use of limiting natural resources and restraint in emissions is the call of the day.

As it is said, ‘Nations don’t build roads. Roads build a nation’. It is very important to develop road infrastructure at a brisk pace, if we want to attain / sustain high economic growth. At the same time, it is equally important to ensure that such infrastructure development is sustainable. Prudent use of limiting natural resources therefore, becomes our responsibility now, more than ever before.

There may be many ways of bringing in sustainability in building roads. Here are some of the ways in which Zydex Nanotechnology contributes to sustainability in road construction:

- Reducing the usage of limiting resources like bitumen, aggregates etc.
- Extending the life cycles of the roads, so as to defer the demand for such resources
- Reducing fuel consumption leading to reduction in emission

Zydex Nanotechnology for roads is a shift in paradigm, from conventional Top-Heavy, Resource-Intensive and Uneconomical road design to a Balanced, Sustainable and Economical road design. It is a ‘Game Changer’ in that sense.

2 CHALLENGES OF CONVENTIONAL ROAD DESIGN AND MITIGATION

Most soils (with an exception of those having a high sand content) are expansive in nature, i.e., they have a tendency of swelling when they become wet with water. The soil when wet, tend to lose their bearing strength and most often cannot be relied upon for taking on the traffic loads. The designers are therefore, forced to design roads such that most of the load is taken by the ‘Structural Layers’ – the stone bases and the asphaltic layers on the top. (Refer to Figure 1)

Figure-1

Conventional Road Design

Top heavy, Resource intensive Uneconomical

Stone Base (WBM / WMM)

Granular Sub Base (GSB)

Compacted Subgrade
The structural layers, i.e. the stone bases and the asphaltic layers not only consume limiting resources like stone and bitumen, but are also considerably more expensive. As we consume more of these resources, they will become even more scarce and the structural layers will become even more expensive in coming days.

Unfortunately, despite such expensive top-heavy designs, we often find undulations and cracks on the top surface of the roads. A probable reason for this could be soil bases not having adequate bearing strength.

Zydex Nanotechnology can treat almost any type of soil (having a CBR > 2) to make it water-resistant. If added with polymers / cement, the soil can be further stabilized to have a CBR of close to 100 or even more. All this can be achieved at a globally affordable economy.

Now, with much higher CBR, the soil bases are ready to participate in sharing the traffic load, taking considerable burden off the structural layers. This presents the designer with an opportunity to optimize his road design, by significantly cutting down the thicknesses of the stone layers and the asphaltic layers. (Refer to Figure-2)

![Optimized Road Design](image)

Figure-2

And to end, in cases where local soils are too expensive for road construction, the designers are forced to specify borrowed soil from long distances. Since almost any type of soil can be treated with Zydex Nanotechnology, it gives the designer a huge opportunity to make the local soil useable and to save cost of bringing soils from long distances.

3 CAPEX REDUCTION – CASE STUDY

We applied the principles of the new changed game to a real case in India, for a rural road at Bellary, Karnataka, India. The two proposed designs and their costs are as below. (Refer to Table-1)
<table>
<thead>
<tr>
<th>ZB 112</th>
<th>112</th>
<th>375</th>
<th>375</th>
<th>ZB 112</th>
<th>262479.8</th>
<th>172</th>
<th>172</th>
<th>390</th>
<th>0.5</th>
<th>2770</th>
<th>2770</th>
<th>2770</th>
<th>2770</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5%</td>
<td>0.5%</td>
<td>3.5%</td>
<td>3.5%</td>
<td></td>
<td>1 kg Termite-chitin in 500 it</td>
<td></td>
<td></td>
<td>35 mm</td>
<td>3.5%</td>
<td>21%</td>
<td>21%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ZB 112</td>
<td>112</td>
<td>375</td>
<td>375</td>
<td>ZB 112</td>
<td>262479.8</td>
<td>172</td>
<td>172</td>
<td>390</td>
<td>0.5</td>
<td>2770</td>
<td>2770</td>
<td>2770</td>
<td>2770</td>
</tr>
<tr>
<td>0.5%</td>
<td>0.5%</td>
<td>3.5%</td>
<td>3.5%</td>
<td></td>
<td>1 kg Termite-chitin in 500 it</td>
<td></td>
<td></td>
<td>35 mm</td>
<td>3.5%</td>
<td>21%</td>
<td>21%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
The comparison shows that the optimized Zydex design saves 18% CAPEX over conventional design.

4 BOND COATS FOR BETTER ADHESION BETWEEN ROAD LAYERS

*Prime Coat* (between stone layer and asphaltic layer) and *Tack Coat* (between two asphaltic layers), collectively called *Bond Coats*, are crucial for structural integrity of a road. If the adhesion between any of the layers is poor, there are possibilities of slippage. The different layers of the road will now behave individually and not as a single unit, due to which, the load transfer will not be uniform. As a result, the top layers are subject to heavy fatigue load and they tend to develop cracks.

The primary reason for failure of conventional bond coats is the mechanism of their bonding. What they create is a weak physical bond between two layers. Secondly, the spray of the bitumen emulsion is often spotty and non-uniform, which leads to uneven load transfer and localized concentrated stresses in the top layers.

Zydex Nanotechnology makes the bond coats penetrative and enables chemical bonding between road layers. With Zydex nanotechnology additives, the bitumen emulsion can now be sprayed fine with much smaller droplets, which not only gives 100% uniform coverage but also makes the sprayed surface waterproofed.

Fine spray and better spreading also enables the designer to significantly cut down the residual bitumen and still have far better performance as compared to the conventional bond coats. (Refer to Figure-3)

Figure-3

Waterproofed Prime coat  Trackless Tack coat

5 ASPHALTIC LAYERS – CHEMICAL BONDING BETWEEN AGGREGATES AND BITUMEN

Most aggregates contain silicates and they have on their surface the ‘OH’ groups or ‘Silanol’ groups. The aggregate surface therefore, has great affinity to water. In other words, the surface is ‘Polar Hydrophilic’ or Water Loving’.

On the other hand, asphalt has only 5-15% of polar components (Asphaltenes). Balance 85-95% of asphalt (Maltenes) is Non-Polar and oil-like. As a result, in the conventional HMA, only 5-15% of asphalt participates in bonding with the aggregates. Hence the coating is highly susceptible to water. The moment it comes into prolonged water contact, the asphalt coating on the aggregates tends to strip.

Zydex Nanotechnology additive to asphalt, namely ZycoTherm, ensures through nano-modification that the aggregate surface is converted to ‘Non-Polar Hydrophobic’ or ‘Asphalt-Loving’ surface and enables chemical bonding between the aggregates and the asphalt. Moreover, now since the surface of the aggregate is non-polar and asphalt loving, 85-95% of asphalt (Maltenes) now participate in bonding with the aggregates. This further improves the bond strength. (Refer to Figure-4)
Figure-4

The concept of chemical bonding between aggregates and asphalt is a new introduction by Zydex Nanotechnology. What prevailed so far was only physical bonding. Hence, the conventional laboratory tests for water susceptibility / strength of hot mix, specified under different standards are grossly inadequate to test chemical bonding under Zydex Nanotechnology.

In order to bring out superiority of chemical bonding over conventional bonding, we decided to go for extension of standard tests, in terms of higher temperatures as well as longer test durations. The extensions we went for are given in the following table. (Refer to Table-2)

Table-2

<table>
<thead>
<tr>
<th>TESTS</th>
<th>CURRENT SPECIFICATIONS</th>
<th>ZYDEX SPECIFICATIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boil Test ASTM D 3625</td>
<td>10 minutes</td>
<td>6 Hours</td>
</tr>
<tr>
<td>Boil Test ASTM D 3625 (4% Salt Water) Zydex Standard</td>
<td>10 minutes</td>
<td>6 Hours</td>
</tr>
<tr>
<td>Static Immersion AASHTO T182</td>
<td>24 Hours</td>
<td>7/15 Days</td>
</tr>
<tr>
<td>Rolling Bottle EN 12697-11</td>
<td>24 Hours</td>
<td>96 Hours</td>
</tr>
<tr>
<td>ITSR EN 12697 23</td>
<td>Conditioning for 72 hours at 40°C</td>
<td>Conditioning for 7 Days at 60°C</td>
</tr>
<tr>
<td>Moisture Resistance AASHTO T 283</td>
<td>1 Freeze-Thaw, 60°C 24 hours conditioning Cycle</td>
<td>5-7 Cycles</td>
</tr>
<tr>
<td>Marshall Compressive Strength AASHTO T 165 / ASTM D 1075</td>
<td>60°C 24 hours</td>
<td>60°C 14 Days</td>
</tr>
</tbody>
</table>

The test results after such extended testing are summarized below.

0.1 wt% addition of ZycoTherm Doped Bitumen showed improvement in coating retention on siliceous aggregates (~95% compared to ~20% for undoped bitumen) in standard 10 minutes boil test.

It has maintained the same performance even after 60 minutes of boil test.
It was observed that ZycoTherm doped binder meets all basic properties of viscosity grade paving bitumen (IS:73-2013).

Further it shows resistance towards temperature and air oxidation compared to undoped binder which is beneficial.

ZycoTherm modified bitumen shows comparable DSR measurements for rutting behaviour and after ageing test shows comparatively better resistance towards fatigue cracking.

The Marshall compressive strength is comparable for modified and unmodified bitumen.

The overall conclusion is ZycoTherm added bitumen helps to improve stripping resistance of binder significantly and improve resistance to oxidation and temperature without affecting strength and flexibility. (Refer to Figure-5)

Figure-5

Summary of the laboratory tests conducted at one of the renowned refinery in India:

- Addition of ZycoTherm in paving grade bitumen (VG 30) doesn’t alter resultant binder properties significantly and is compatible with paving grade bitumen.
- ZycoTherm doped bitumen improves anti-stripping property of the binder.
- ZycoTherm doped bitumen shows resistance towards air oxidation.
- The DSR studies suggest that the addition of ZycoTherm does not alter or modify original binder properties in terms of rutting and/or fatigue cracking.
- The Marshall Stability studies indicate that the mixes show comparable compressive strength.

It was observed that ZycoTherm Doped Bitumen withstood even the severely extended tests up to 7 times, indicating that the pavement life can be extended 2-3 times by using Zydex Asphalt additive Doped Bitumen.

Moreover, addition of ZycoTherm to asphalt also allows lower mixing temperature (by 10 to 15 °C) and lower laying / compaction temperature (by 30-35 °C), saving 7-10 % fuel and emissions.
6 CONCLUSION

In summary, here is what Zydex Nanotechnology for roads has to offer.

SOIL LAYERS

Zydex Nanotechnology makes the soil moisture resistant, reduces expansiveness and stabilizes the soil to improve its bearing strength manifold. If used with 1% cement, it can stabilize almost any type of soil, by improving the California Bearing Ratio (CBR) to even 100 or above.

Here is the real change in game, as stronger soil base would now allow optimization of road section design with lower thicknesses of expensive top layers, potentially saving 10-15% of road construction cost.

BOND COATS

Prime & Tack coats become 100 % waterproofed, due to penetration and chemical bonding. This also ensures uniform load transfer. And all this at lower residual bitumen.

ASPHALTIC LAYERS

Chemical bonding between aggregates and asphalt eliminates moisture induced damage to asphaltic layers. Efficient coating, better workability and compaction at lower temperatures, reduces fuel consumption by 7-10 %.

The future Green Roads can be built by local soils and convert them to become water resistant / Impermeable and stronger with chemically bonded Bond Coats and bitumen mixes.

Thus, the game changing Zydex Nanotechnology significantly reduces CAPEX, maintenance cost and overall life cycle cost of roads, while contributing to sustainable development, paving the way to a greener tomorrow.

*****
Material Characteristics of Polypropylene-Coated Multifilament Glass-Fibre Reinforced Hot-Mix Asphalt Mixtures

Dr Pyeong Jun Yoo¹, Dr Tae Woo Kim²

¹Korea Institute of Civil Engineering and Building Technology, Ilsan, Goyang, Korea
E-mail for correspondence: pjyoo@kict.re.kr

²Korea Institute of Civil Engineering and Building Technology, Ilsan, Goyang, Korea
E-mail: twkim@kict.re.kr

1 INTRODUCTION

This study presents mechanical characteristics of thermoplastic polymer-coated glass fibre rod and glass fibre scrap-reinforced hot-mix asphalt (HMA) mixtures. The toughening effects of the reinforced HMA mixtures were characterised by using the results from indirect tensile loading tests and Hamburg wheel-tracking tests.

The performance of HMA mixtures reinforced with various fibres, such as carbon fibres or polyethylene terephthalate fibres has been reported to be superior to the mechanical behaviors of general HMA in terms of toughness, indirect tensile strength, shear strength, and fracture energy. The improved toughness and fracture energy, which may increase the fatigue life of HMA, were the representative effects in the use of those fibers with HMA (Fitzgerald 2000, Yoo & Kim 2014).

A promising cause of toughening may be that the fibers in HMA can enhance the shear strength at the interface between the HMA matrix and fibers, and the enhanced property can delay the initiation and propagation of damages. Yoo et al. proposed a reinforcing mechanism and as well as a way of determining optimizing fiber content utilizing a direct tensile loading test developed by them. However, the fiber’s dimension and aggregate gradation in their study were fixed in constant values so that the applicability was to be marginal (Li et al. 1991, Yoo & Kim 2014).

In view of several studies emphasized that not all fibers mixed in the composite are equally effective in their toughening effects, the fibre’s random distribution and orientation features require inevitable assumptions regarding the probability density functions for calculating the effective composite stresses along the failure plane by accounting for the fiber’s bridging forces. The randomness may be overcome by the acceptable dispersion of fibres without fibre’s balling by deciding the effective dimension or the optimum content of fibres in HMA (Yoo & Kim 2014).

Recently, non-synthetic and synthetic fibres have been utilized to improve the performance of Hot-Mix Asphalt (HMA) mixture against permanent deformation and fatigue cracking. Few researches reporting on experiments using various fibres in asphalt concrete mixtures have found in the literatures.

Kaloush et al. found that the use of polypropylene (PP) and aramid fibers mixed in the asphalt mixture improves the resistance to shear deformation, which is verified by the triaxial shear strength tests. The fibrous mixture results in an increase of 25–50% for the tensile strength (Kaloush et al. 2013).

Lee et al. showed that the increase in fracture energy due to the addition of recycled carpet Nylon fibers represents a potential for improving the fatigue life of asphalt mixture. Their study also addressed that the fiber’s balling while mixing should be overcome to ensure the reinforcing effect on asphalt concrete mixture (Lee et al. 2005).

Mahrez and Karim reported that the addition of glass fibres in a constant length of 20 mm resulted in higher resistance to the fatigue cracking and permanent deformation by the repeated load indirect tensile tests. They concluded that the chopped glass fibres distributed in random directions in a mixture resist the shear displacement and prevent the internal dislocation of aggregates effectively (Mahrez & Karim 2007).
Although the effect of fibres in asphalt concrete can be very different depending on the dimensions and contents of fibers, this study only utilizes a constant fiber’s dimension and a fixed mix design to verify the marginal strengthening effect of glass fibres in asphalt concrete. Besides the fibrous mixture, it is worth noting that the fibres can affect the ductility or stiffness of bitumen itself due to addition of fibres. However, this study only addresses some experimental results of including glass fibers in asphalt concrete mixtures.

The objective of this study was to evaluate the mechanical characteristics of the glass fibre rod and glass fiber scraps, those are coated by the PP resin, -reinforced asphalt mixtures using the indirect tensile loading and Hamburg wheel tests. The manufacturing process for the PP-coated glass fibre rod and glass fiber scraps was proposed to obtain an effective fibre’s dispersion without any fiber’s balling in HMA by increasing the specific gravity and dimension of glass fibres and scraps as similar as the natural aggregates.

2 MULTIFILAMENT GLASS FIBER-REINFORCED HMA

The glass fibres used in this study were a chopped glass fiber rod containing 800 to 1,000 monofilaments of glass fibres. The roved multifilament glass fiber was first coated by the PP resin through the impregnation process. Besides the physical characteristics, as shown in Table 1, the mechanical properties of the fibres, such as tensile strength and Young’s modulus are referred from the study of Wallenberger et al., are at least 100 times higher than the typical values of a HMA at room temperature (Wallenberger et al. 2001).

The density of glass fibre is relatively comparable to a general aggregate greater than 2.0 so that the multifilament fiber rod may behave as an aggregate without any noticeable fibre’s balling during and after the dry and wet mixing process of HMA. The effective dispersion without fibre’s balling may be expected by the aggregate-like behavior of the fiber.

Table 1. Physical characteristics of Glass Fibres

<table>
<thead>
<tr>
<th>Multifilament Glass Fiber</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (g/cm³)</td>
<td>2.55</td>
</tr>
<tr>
<td>Elongation (%)</td>
<td>4.5–5.0</td>
</tr>
<tr>
<td>Number of Filament</td>
<td>800–1,000</td>
</tr>
<tr>
<td>Length (mm)</td>
<td>10–12</td>
</tr>
</tbody>
</table>

Two different contents of multifilament fibers, such as 1% and 2% to the whole weight of mixture were used to prepare specimens for the lab tests. Only one specimen among them contains the fiber-scraps in a shape of aggregate; the fiber-scraps were coated by the PP resin first through an impregnation process as the glass fiber rod did, as shown in Figure 1a and 1b. The glass fiber scraps are a by-product while making a roving fiber. The maximum sieve size of the scraps should be 75μm, which is a sieve size of mineral filler in general. The scraps passed 75 μm-sieve were utilized in making the aggregate-like scraps in this study, those are first coated by the PP resin (PPGS) and cut into the predefined dimension at the last step of an extrusion process, as shown in Figure 1b.

Figure 1 (a) Glass fiber scrap (b) Aggregate-like PP coated scrap (PPGS)

The gradation of scraps is as shown in Table 3. Because of the difficulty in control the contents of dust or mineral filler when doing a mix design in an asphalt plant, this study proposed the PPGS as a mineral filler substitute material to make the mixing with aggregate more easily than the powder type and to replace all the mineral fillers and dusts in the Hot-Mix asphalt concrete.
Besides the PP-coated glass fibre scrap (PPGS), the PP-coated multifilament glass fibres rod (PPGF) were developed for the use of HMA application as reinforcing media, as shown in Figures 2a and 2b. The extrusion process with an impregnation step for the PP coating, Figure 2a, can make the PPGF in an elliptical shape of cross-section. The PPGF were cut into 10 mm-long and 1 mm wide (short side) and 2 mm wide (long side), Figure 2b.

In addition, the PP resin particles in the circular shape, Figure 2b, were added into HMA according to the equivalent weight ratio between the glass fiber before coating and the PP resin. The PP particles were fully melted down during the dry and wet-mixing process with hot aggregate and binder in a lab or plant and may play a role as an adhesion improver. Figure 2b shows the PPGF including the PP particles.

### Table 2. Gradation of glass fiber scraps

<table>
<thead>
<tr>
<th>Sieve Size (μm)</th>
<th>% Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>100.0</td>
</tr>
<tr>
<td>75</td>
<td>83.0</td>
</tr>
<tr>
<td>45</td>
<td>47.0</td>
</tr>
</tbody>
</table>

Figure 2 (a) Extrusion and impregnation process (d) PP Coated glass fiber and PP particles

Table 3 shows the fiber contents and the coarse-side aggregate gradations used in this study. Except the control specimens without fibers in Table 3, the contents of multifilament fibre varied from 1% to 2% of the total weight of HMA and only one specimen contains the PPGS. According to the mixture design, one control specimen and three fibrous specimens were tested to address the relative toughening effect due to inclusion of fibres through the indirect tensile strength and Hamburg Wheel Tracking tests.

### Table 3 Fiber contents and aggregate gradation

<table>
<thead>
<tr>
<th>Asphalt Content (%)</th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>PPGF (%)</td>
<td>0</td>
<td>1</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>PPGS (%)</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sieve Size (mm)</th>
<th>% Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.45</td>
<td>5.45</td>
</tr>
<tr>
<td>19</td>
<td>100.0</td>
</tr>
<tr>
<td>13</td>
<td>93.0</td>
</tr>
<tr>
<td>10</td>
<td>56.0</td>
</tr>
<tr>
<td>4.75</td>
<td>31.0</td>
</tr>
<tr>
<td>2.36</td>
<td>17.0</td>
</tr>
<tr>
<td>1.18</td>
<td>10.0</td>
</tr>
<tr>
<td>0.6</td>
<td>7.0</td>
</tr>
<tr>
<td>0.3</td>
<td>4.0</td>
</tr>
<tr>
<td>0.15</td>
<td>2.0</td>
</tr>
<tr>
<td>Filler</td>
<td>2.0</td>
</tr>
</tbody>
</table>

The aggregate gradations were designed to have the air voids in between 10 to 13%. The air void for the plain mixture resulted in around 10%, and the fibrous mixtures resulted in about 13%, which is higher air void than the plain mixture, because the addition of fibre makes the mixture’s volume to be increased.
The fixed asphalt content of 5.45% was utilized as a predefined value for a coarse aggregate gradation within the context of this study through the Marshall mix-design process referred from other study (Yoo & Kim 2014).

This study attempted to avoid the binder content’s dependency on the mixture’s behavior, aiming to observe only variations due to the fiber contents. The asphalt binders for all of the mixtures used in this study were of Superpave PG 64-22 grade with a penetration of 65 and a softening point of 49 °C.

![Dry-mixing fibers and aggregate](image1.png)  ![Dispersed fibers in HMA after wet-mixing](image2.png)

Figure 3 (a) Dry-mixing fibers and aggregate, (b) Dispersed fibers in HMA after wet-mixing

Before adding the asphalt binder, the dry mixing under the temperature around 170°C, Figure 3a, was performed to check whether the fibres were entangled or not. All the PP, such as the additional PP particles and impregnated PP in PPGF, was melted down while doing the dry mixing during 2 minute because of the relatively lower melting temperature around 150°C of PP. The impregnated fibres were released in each monofilament and dispersed in the whole area of the mixtures, as shown in Figure 3b.

Without any balling of the fibers in the mixture, all of the specimens were compacted in a Marshall mold at approximately 160°C until the target 75-tamps on both side for each specimen was achieved to perform the indirect tensile loading tests.

On the other hand, all of the specimens for the Hamburg wheel tests were compacted in a 150-mm-diameter mold at approximately 160°C using a gyratory compacter, with 100-110 gyrations applied to each specimen until the target height for each specimen (i.e., 60 mm) was achieved. Two specimens with dimensions of approximately 150 mm (Diameter) × 60 mm (thickness), were set into the Hamburg test mold.

3 LABORATORY TESTS

3.1 Indirect Tensile loading Test

The toughening effect of glass fiber fiber-reinforced HMA is characterized by considering specimens subjected to indirect tensile loading according the AASHTO T-283-07 with a freeze-thaw cycle.

Before performing the freeze-thaw cycle, the indirect tensile loading tests without the freeze-thaw cycle were performed to estimate the indirect tensile strengths for all the specimens in Table 2. The GF-2-2% (PPGF 2 % and PPGS 2 %) in Figure 4 resulted in the highest indirect strength around 1.0 MPa, which is almost twice higher strength comparing to the value of plain specimen. In addition, to estimate the moisture susceptibility, the GF-2-2% was selected for performing the freeze-thaw cycle.

![Indirect tensile strength of dry specimens](image3.png)

Figure 4 Indirect tensile strength of dry specimens
Test specimens, such as GF-0% (No Fiber) and GF-2-2% (PPGF and PPG) in Figure 4, were only conditioned according to the AASHTO T-283-0 procedure. Figures 5a and 5b illustrate the indirect tensile loading test set-up, six specimens for each test, half of them to be tested dry condition and the other half to be tested after partial saturation and moisture conditioning with a freeze-thaw cycle (AASHTO 2011).

Figure 5 (a) Indirect tensile loading test, and (b) Specimens in conditioning

The tensile strength and the tensile strength ratio are calculated as follows:

\[ S_t = \frac{2008 \times P}{\pi t D} \]  

(1)

where, \( S_t \) = tensile strength (kPa), \( P \) = Maximum Load (N), \( t \) = specimen thickness (mm) and \( D \) = specimen diameter (mm).

\[ TSR = \frac{S_2}{S_1} \]  

(2)

where, \( S_1 \) = average tensile of the dry subset (kPa), \( S_2 \) = average tensile of the conditioned subset (kPa).

The tensile strength of the glass fibrous specimen in dry condition, Figure 6a, resulted in 976.8 kPa and 689.2 kPa for the specimen without fiber. The tensile strength in wet conditions, Figure 6b, resulted in 712.2 kPa for the glass fibrous specimen and 304.3 for the plain specimen.

Figure 6 (a) Dry condition (b) Freeze-thaw condition

The TSR according to the Equation 2, Figure 7, for the fibrous specimen was about 0.73 and 0.44 for the specimen without fiber. The indirect strength and the TSR for the fibrous specimen were almost two times higher values than those from the plain specimen. This may represent that the lower moisture susceptibility and the superior field performance of fibrous asphalt mixture.

Figure 7 TSR comparison
3.2 Hamburg wheel tracking test

The Hamburg wheel test is still believed to be the best promising tool to estimate the resistance to the rutting and stripping. In a dual wheel track system, two loaded steel wheels track over the samples in a heated water bath, generally hold at 50°C, the deformations are recorded versus the number of loading passes.

Hamburg wheel test needs to be allowed some room for variability due to difficulties in maintaining consistencies, such as materials, sampling and test conditions. However, it is an effective test to compare the resistance of rutting or stripping for two different mixtures simultaneously (Lu & Harvey 2006a).

The dimensions of cylindrical specimens are 150±2 mm in diameter and 62±2 mm in height, as shown in Figures 8a–8d. The test criteria for allowable maximum rut-depth of 12.5 mm depending on the types of binders, such as PG 64, PG 70 or PG 76 set the number of wheel passes at 10,000, 15,000 and 20,000 respectively (Rahman & Hossain 2014).

After completed the tests, the total rut-depth of the fibrous specimen, Figure 8a, was only 4.59 mm until completed the number of wheel passes of 20,000. On the contrary, all the control specimens without fibers, Figures 8(b), 8(c), and 8(d), were totally damaged before 5,000 wheel passes.

![Figure 8 Hamburg wheel track test specimens (a) Fibrous Specimens, (b) Normal control 1, (c) Normal control 2, and (b) Normal control 3](image)

The Rut-Depth versus number of wheel passes graph, Figure 9, represents that the fibrous specimen can support the wheel loading until the 20,000 loading repetitions results in its terminal rut-depth of 4.59 mm, which is very low value of rut-depth indicating that there is no damage or stripping initiation. This result may indicate that the shear strength of the fiber reinforced mixture was enhanced by the interfacial bond strengthening effect of the fibers along the vertical failure plane of the specimen.

![Figure 9 Rut-Depth vs. Number of wheel passes](image)

4 CONCLUSION

This study presented experimental results of polypropylene-coated glass fiber (PPGF) and aggregate-like glass fiber scraps (PPGS) reinforced hot-mix asphalt (HMA) mixtures.

The glass fibres rod used in this study were a chopped strand containing 800 to 1,000 mono-filament of glass fiber. The multfilament glass fiber was coated by the PP resin through the impregnation process then cut into a proper length such as 10 mm in this study.
The glass fiber scraps as the mineral filler substitute are the by-product while making the roving glass fiber. The maximum sieve size of the scraps should be 75μm, which is a sieve size of mineral filler in general. The scraps passed 75 μm-sieve were utilized in making the aggregate-like scraps, those were coated by the PP resin through the impregnation step during the extrusion process.

The toughening effects of the glass fibre rod and scrap reinforced HMA mixtures were characterized using the indirect tensile loading tests and Hamburg wheel tracking tests.

The indirect tensile loading tests were performed to calculate the relative indirect strengths of samples and to compare the moisture susceptibilities of them. The indirect strength and the TSR of fibrous specimens are almost two times higher than the results from the plain specimen. This would represent that the lower moisture susceptibility and the superior field performance of fibrous asphalt mixture.

Accelerated rutting tests using the Hamburg wheel test set-up for the fibrous and the plain mixtures were performed. The rut-depth of the fibrous mixture was 4.59 mm until completed the 20,000 loading repetitions. The rut-depth for the plain mixtures without fibres resulted in 15–20 mm before completing the loading passes of 5,000. This may mean that the fiber’ bridging effect could resist the tensile or compressive movement of the fibrous mixture even on the rigid plastic foundation of Hamburg wheel test set-up. The rut-depth of the plain HMA resulted in at least three times higher than the fibre-reinforced HMA did.

This result indicates that the tensile or compressive strength of the fiber reinforced mixture was enhanced by the interfacial bond strengthening effect of the fibers along the failure plane. The fibrous HMA could prolong the performance life of the pavement system due to the bridging effect of fibres.

ACKNOWLEDGEMENTS

This study was supported by the Internal Research Program (Pothole-Free Pavement System) of Korea Institute of Civil Engineering and Building Technology, funded by the Ministry of Knowledge and Economy.

REFERENCES


FLOW NUMBER PROPERTIES OF STONE MATRIX ASPHALT IN INDONESIA

TRACK

AUTHOR (Capitalize Family Name) POSITION ORGANIZATION COUNTRY

Nyoman SUARYANA Head of Experimental Station for Material and Road Pavement Institut of Road Engineering (IRE) Indonesia

CO-AUTHOR(S) (Capitalize Family Name) POSITION ORGANIZATION COUNTRY

Bambang Sugeng SUBAGIO Lecturer Bandung Institute of Technology Indonesia

E-MAIL (for correspondence) nyomansuaryana@yahoo.com; nyoman.suaryana@pusjatan.pu.go.id

KEYWORDS: stone matrix asphalt, asbuton, cellulose, flow number, rutting

ABSTRACT:
Rutting has been considered the most serious distress in flexible pavement for many years. One type of asphalt paving are developed to be more resistant to rutting is the SMA (Split Mastic Asphalt or Stone Matrix Asphalt), and flow number is an explanatory index for the evaluation of the rutting potential of asphalt mixtures. The objective of this study was to analysis of flow number properties of Stone Matrix Asphalt (SMA) in some different variables such as testing temperature, binder type, mix volumetric and mix grading. In this study, the flow number test conducted without confining pressure with axial stress of 600 kPa (87 psi). The duration of the load pulse is 0.1 sec. followed by a rest period of 0.9 sec. While the temperature used vary from 20, 35, 45 and 56 °C. Two different binder type was used, i.e. pure petroleum bitumen 60/70 pen grade and pure petroleum bitumen modified by granular Buton rock asphalt (asbuton). Percentage of aggregate passes the sieve number 200 (filler) and void in mix also varied. The addition of granular asbuton in the SMA mix has a significant effect on improving the rutting resistance, and granular asbuton can behave as a stabilizer and complimentary filler in the SMA mix. The flow number (rutting resistance) of SMA mixture observed in this study were decreases when the temperature and void in mix increases. The addition of the filler material on a certain boundaries will also increase the resistance to rutting.
FLOW NUMBER PROPERTIES OF STONE MATRIX ASPHALT IN INDONESIA

Nyoman Suaryana¹, Bambang Sugeng², Djunaedi Kosasih², Sjahdanulirwan¹

¹Institute of Road Engineering; Jalan AH Nasution no.264; Bandung 40294; Indonesia
²Bandung Institute of Technology; Jalan Ganeca 10; Bandung 40132; Indonesia

Email for correspondence: nyomansuaryana@yahoo.com

1 INTRODUCTION

Rutting in flexible pavement caused by a combination of densification or one-dimensional compression and lateral movement or plastic flow of materials. The result of investigations on the 15 mixtures used in Strategic Highway Research Project (SHRP) show deformation due to plastic flow of materials much more dominant than the results of compression (Long, 2001).

In NCHRP Project 9-19, the flow number correlated well with the rutting resistance of mixtures as shown in experimental sections at the FHWA Pavement Testing Facility, MNRoad, and WesTrack (Witzak, 2002). The flow number test is a permanent deformation test that has been used by researchers since the 1970’s to measure the rutting potential of asphalt concrete mixtures (Witzak, 2007).

Haversine axial compressive-load pulses are applied to the specimen. The duration of the load pulse is 0.1 sec followed by a rest period of 0.9 sec. The test may be conducted with or without confining pressure. The variation introduced by the NCHRP Project 9-19 research is the concept of flow number, which is defined as the number of load pulses when the minimum rate of change in permanent strain occurs during the repeated-load test (Bonaquist, 2012, Witzak, 2002).

The Arizona State University suggested using the model of the Francken algorithm instead of finite difference to make the relationship between the number of cycle with permanent strain, which is as follows:

\[ \varepsilon_p = a N^b + c (e^{dN-1}) \]  

where:

- \( \varepsilon_p \) = permanent strain, %
- \( N \) = number of cycle
- \( a, b, c, d \) = fitting parameters

The flow number is then determined from the second derivative of the fitted curve. The flow number is the number of cycles were the second derivatives change from negative to positive. According to Rodezno (2010), the value of flow number (Fn) of hot-mix asphalt is influenced by mix volumetric properties, binder type, stress condition, testing temperature and mix gradation.

One type of asphalt paving are developed to be more resistant to rutting is the SMA (Split Mastic Asphalt or Stone Matrix Asphalt). The SMA has been known since the mid 1960s, Dr Zichner was its inventor. SMA consisted of creating a very strong aggregate skeleton of coarse aggregates and filling the spaces between them with mastic (i.e., a mix binder, filler, and sand). SMA mixtures require a high content of binder, which results in thick binder films on the aggregate grains. To avoid the draindown effect, stabilizing additives are indispensable in most cases. The two main technique of reducing binder draindown are added additive that absorb part of the binder (such as cellulose, mineral fiber, textile) or added additive that increase binder viscosity (such as polymers) (Blazejowski, 2011)

The objective of this study was to analysis of flow number properties (rutting resistance) of Stone Matrix Asphalt (SMA) in some different variables such as testing temperature, binder type, mix volumetric and mix grading. The additive to prevent binder draindown was used in this study are cellulose as binder absorbers and natural rock asphalt (asbuton) as viscosity boosters.
2 EXPERIMENTAL STUDY

2.1 Selected Materials

The Basalt aggregate used in this study prepared from Sewo quarry, West Java Indonesia. Several tests has been conducted on aggregate particles, Table 1 shows the properties related to aggregate used in this study.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>- L. A. Abrasion</td>
<td>16.91</td>
<td>%</td>
</tr>
<tr>
<td>- Sand Equivalent</td>
<td>-</td>
<td>74.2</td>
</tr>
<tr>
<td>- Specific Gravity</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Bulk</td>
<td>2.657</td>
<td>-</td>
</tr>
<tr>
<td>- SSD</td>
<td>2.695</td>
<td>2.706</td>
</tr>
<tr>
<td>- Apparent</td>
<td>2.762</td>
<td>2.758</td>
</tr>
<tr>
<td>- Absorption</td>
<td>1.434</td>
<td>1.092</td>
</tr>
<tr>
<td>- Percentage of fractured particles</td>
<td>100/100</td>
<td>-</td>
</tr>
<tr>
<td>- Soundness</td>
<td>1.04</td>
<td>1.54</td>
</tr>
</tbody>
</table>

The aggregate gradation for SMA has been chosen based on AASHTO M 325-08. Aggregates grading can be characterized as one of the factors that influence the resistance of asphalt mixtures to rutting. The middle grading and the nominal maximum aggregate size of 12.5 mm was chosen in this study, as shown in figure 1.

![Figure 1. Grading of SMA mix](image)

Bitumen is only a minor component of bituminous mixes. But it has a crucial part to play in providing viscos–elasticity and acting as a durable binder. The primary or routine rheological properties are penetration, softening point and viscosity. Table 2 Table 2. Rheology properties of bitumen LGA and pure petroleum bitumen 60/70 pen grades showed that the bitumen content of bitumen from granular asbuton with maximum size 1.18 mm is 23 % and its bitumen is relatively hard as indicated by penetration value 41 dmm compare with the petroleum bitumen 60/70 by penetration value 62 dmm. The softening point of the bitumen of asbuton is also higher than the petroleum bitumen, which are 58 °C and 50 °C respectively. However at fact the bitumen of asbuton can not be separated from its mineral during the mixing process in the Asphalt Mixing Plant (AMP) which is taken in short time.
The temperature susceptibility is usually described as the change of primary or routine rheological properties of bitumen with temperature. Pleiffr and Van Dormaal defined the temperature susceptibility of bitumen as the Penetration Index. The value of PI ranges from -3 for highly temperature susceptible bitumen to about +7 for highly blown low temperature susceptible (high PI) bitumen.

Based on the data in Table 2 and using equation Pleiffr and Van Dormaal, the Penetration Index (PI) of Granular Asbuton is +0.17 and petroleum bitumen 60/70 pen grade is -0.61. The PI indicate that asbuton bitumen have low temperature susceptible compare than petroleum bitumen pen 60.

<table>
<thead>
<tr>
<th>Property</th>
<th>Bitumen of Granular Buton Rock Asphalt (Asbuton)</th>
<th>Petroleum Bitumen 60/70 pen grade</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Penetration at 25 °C</td>
<td>41</td>
<td>62</td>
<td>0.1 mm</td>
</tr>
<tr>
<td>Softening Point (R &amp; B)</td>
<td>58.1</td>
<td>50.3</td>
<td>°C</td>
</tr>
<tr>
<td>Ductility</td>
<td>&gt; 140</td>
<td>&gt; 140</td>
<td>cm</td>
</tr>
<tr>
<td>Solubility in C\textsubscript{2}HCl\textsubscript{3}</td>
<td>-</td>
<td>99.5</td>
<td>%</td>
</tr>
<tr>
<td>Flash Point</td>
<td>-</td>
<td>317</td>
<td>°C</td>
</tr>
<tr>
<td>Unit Weight</td>
<td>1.125</td>
<td>1.035</td>
<td>-</td>
</tr>
<tr>
<td>Loss on Heating TFOT</td>
<td>3.96</td>
<td>0.0041</td>
<td>%</td>
</tr>
<tr>
<td>Penetration after loss on heating</td>
<td>27.8</td>
<td>47.12</td>
<td>0.1 mm</td>
</tr>
<tr>
<td>Softening Point after loss on heating</td>
<td>68.6</td>
<td>51.3</td>
<td>°C</td>
</tr>
<tr>
<td>Ductility after loss on heating (TFOT)</td>
<td>25</td>
<td>&gt; 140</td>
<td>cm</td>
</tr>
<tr>
<td>Bitumen content</td>
<td>23.3</td>
<td></td>
<td>%</td>
</tr>
</tbody>
</table>

2.2. Sample Preparation

The optimum bitumen content were determined based on volumetric properties according to AASHTO M325 requirement, i.e. Void in Mix (VIM) of 4 %, Void in Mineral Aggregate (VMA) minimum of 17 % and bitumen draindown maksimum of 0.3 %.

The middle limit grading and compactive effort of 2 x 75 blows in Marshall were used in selecting the binder content. Two different SMA mixtures type was used, i.e. SMA mixture with cellulose fibers of 0.2 % as stabilizer and SMA mixture with granular asbuton of 7.5 % as stabilizer (referred to as SMAB). Granular asbuton have maximum size of 9.5 mm and almost 60 % was passing no. 200 sieves. Granular Asbuton is expected to serve as a stabilizer and simultaneously as additional filler. Table 3 shows bitumen content of the SMA mix and SMAB mix.

<table>
<thead>
<tr>
<th>Mix Criteria</th>
<th>Specification AASHTO D: M 325-08</th>
<th>SMA added 0.2 % Cellulose fibers</th>
<th>SMAB added 7.5 % granular asbuton, without cellulose</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bitumen Content</td>
<td>6.0 min</td>
<td>6.05</td>
<td>6.54</td>
<td>%</td>
</tr>
<tr>
<td>Void in Mix (VIM)</td>
<td>4.0</td>
<td>4.0</td>
<td>4.0</td>
<td>%</td>
</tr>
<tr>
<td>Void in Mineral Aggregate (VMA)</td>
<td>17.0 min</td>
<td>17.1</td>
<td>17.9</td>
<td>%</td>
</tr>
<tr>
<td>Void Filled Bitumen</td>
<td>-</td>
<td>76.5</td>
<td>76.8</td>
<td>%</td>
</tr>
<tr>
<td>Marshall Stability (kg)</td>
<td>-</td>
<td>634</td>
<td>651</td>
<td>kg</td>
</tr>
<tr>
<td>Flow (mm)</td>
<td>-</td>
<td>5.4</td>
<td>4.1</td>
<td>mm</td>
</tr>
<tr>
<td>VCA mix</td>
<td>Less than VCA &lt; VCA\textsubscript{DRC}</td>
<td>VCA mix = VCA\textsubscript{DRC}</td>
<td>VCA mix = 0.82 VCA</td>
<td>%</td>
</tr>
<tr>
<td>Draindown</td>
<td>0.3 max</td>
<td>0.18</td>
<td>0.22</td>
<td>%</td>
</tr>
</tbody>
</table>

2.3. Testing Apparatus

Flow Number (Fn) test were performed using The Asphalt Mixture Performance Tester (AMPT) device in accordance with AASHTO TP 79-12. Testing performed on 100-mm diameter by 150-mm tall. The flow number test
conducted without confining pressure with axial stress of 600 kPa (87 psi). The duration of the load pulse is 0.1 sec. followed by a rest period of 0.9 sec. The test were continued to 10,000 cycles or until a permanent strain in the SMA specimen reached 5 percent, whichever came first.

### 3 RESULTS AND DISCUSSION

The typical relationship between number of cycle with permanent strain and the relationship between the numbers of cycle with permanent strain rate is shown in Figure 2. The flow number is the number of cycles where the permanent strain rate is minimum. In Table 4 and Table 5, the effects of testing temperature, void ini mix and mix grading on flow number / rutting resistance are presented.

![Figure 2. Typical Result of Flow Number Test](image)

#### Table 4. Flow Number (rutting resistance) of SMA mix

<table>
<thead>
<tr>
<th>Flow Number</th>
<th>Variabel</th>
</tr>
</thead>
<tbody>
<tr>
<td>No.</td>
<td>Testing Temperature (°C)</td>
</tr>
<tr>
<td>Fn (AMPT) (cycle)</td>
<td>(°F)</td>
</tr>
<tr>
<td>1</td>
<td>7500</td>
</tr>
<tr>
<td>2</td>
<td>596</td>
</tr>
<tr>
<td>3</td>
<td>263</td>
</tr>
<tr>
<td>4</td>
<td>72</td>
</tr>
<tr>
<td>5</td>
<td>6176</td>
</tr>
<tr>
<td>6</td>
<td>122</td>
</tr>
<tr>
<td>7</td>
<td>43</td>
</tr>
<tr>
<td>8</td>
<td>10000</td>
</tr>
<tr>
<td>9</td>
<td>187</td>
</tr>
<tr>
<td>10</td>
<td>49</td>
</tr>
<tr>
<td>11</td>
<td>10000</td>
</tr>
<tr>
<td>12</td>
<td>414</td>
</tr>
<tr>
<td>13</td>
<td>220</td>
</tr>
<tr>
<td>14</td>
<td>10000</td>
</tr>
<tr>
<td>15</td>
<td>41</td>
</tr>
<tr>
<td>16</td>
<td>7</td>
</tr>
</tbody>
</table>
Table 5. Flow Number (rutting resistance) of SMAB mix

<table>
<thead>
<tr>
<th>Flow Number No.</th>
<th>Flow Number Fn (AMPT)</th>
<th>Testing Temperature (°C)</th>
<th>Void in Mix (%)</th>
<th>Retain 3/4 (%)</th>
<th>Retain #4 (%)</th>
<th>Passing #200 (%)</th>
<th>σ1 (psi)</th>
<th>σ3 (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10000</td>
<td>20</td>
<td>3.90</td>
<td>0</td>
<td>72.5</td>
<td>9.5</td>
<td>87</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>1758</td>
<td>35</td>
<td>3.90</td>
<td>0</td>
<td>72.5</td>
<td>9.5</td>
<td>87</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>886</td>
<td>45</td>
<td>3.90</td>
<td>0</td>
<td>72.5</td>
<td>9.5</td>
<td>87</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>316</td>
<td>56</td>
<td>3.90</td>
<td>0</td>
<td>72.5</td>
<td>9.5</td>
<td>87</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>10000</td>
<td>20</td>
<td>5.27</td>
<td>0</td>
<td>72.5</td>
<td>9.5</td>
<td>87</td>
<td>0</td>
</tr>
<tr>
<td>6</td>
<td>264</td>
<td>45</td>
<td>5.27</td>
<td>0</td>
<td>72.5</td>
<td>9.5</td>
<td>87</td>
<td>0</td>
</tr>
<tr>
<td>7</td>
<td>49</td>
<td>56</td>
<td>5.27</td>
<td>0</td>
<td>72.5</td>
<td>9.5</td>
<td>87</td>
<td>0</td>
</tr>
<tr>
<td>8</td>
<td>10000</td>
<td>20</td>
<td>4.48</td>
<td>0</td>
<td>72.5</td>
<td>9.5</td>
<td>87</td>
<td>0</td>
</tr>
<tr>
<td>9</td>
<td>372</td>
<td>45</td>
<td>4.48</td>
<td>0</td>
<td>72.5</td>
<td>9.5</td>
<td>87</td>
<td>0</td>
</tr>
<tr>
<td>10</td>
<td>138</td>
<td>56</td>
<td>4.48</td>
<td>0</td>
<td>72.5</td>
<td>9.5</td>
<td>87</td>
<td>0</td>
</tr>
<tr>
<td>11</td>
<td>10000</td>
<td>20</td>
<td>1.07</td>
<td>0</td>
<td>65</td>
<td>11</td>
<td>87</td>
<td>0</td>
</tr>
<tr>
<td>12</td>
<td>1014</td>
<td>45</td>
<td>1.07</td>
<td>0</td>
<td>65</td>
<td>11</td>
<td>87</td>
<td>0</td>
</tr>
<tr>
<td>13</td>
<td>562</td>
<td>56</td>
<td>1.07</td>
<td>0</td>
<td>65</td>
<td>11</td>
<td>87</td>
<td>0</td>
</tr>
<tr>
<td>14</td>
<td>10000</td>
<td>20</td>
<td>6.54</td>
<td>0</td>
<td>80</td>
<td>8</td>
<td>87</td>
<td>0</td>
</tr>
<tr>
<td>15</td>
<td>227</td>
<td>45</td>
<td>6.54</td>
<td>0</td>
<td>80</td>
<td>8</td>
<td>87</td>
<td>0</td>
</tr>
<tr>
<td>16</td>
<td>74</td>
<td>56</td>
<td>6.54</td>
<td>0</td>
<td>80</td>
<td>8</td>
<td>87</td>
<td>0</td>
</tr>
</tbody>
</table>

The bitumen is one of the fundamental component of asphalt mixtures and is used as a cohesive materials to bond the aggregates. Rutting resistance of asphalt mixtures is significantly affected by the stiffness of the bitumen. The addition of gravel as a bitumen is have 0f 41 dm3 mix grade and softening point (R&B) of 58 °C (as shown in Table 2) were increase the stiffness of bitumen. Data obtained as shown in Table 4 and Table 5, shows the phenomenon, where the flow number of SMAB mix are higher than the flow number of SMA mix affected by increasing the stiffness of bitumen. Thats indicates that SMAB (SMA mix with gravel as bitumen as stabilizer and additional filler) have better rutting resistance compare then SMA (SMA conventional using cellulose as stabilizer).

Figure 3, Figure 4 and Figure 5 describe the effect of temperature, percentage filler and void in mix on rutting resistance of the asphalt mixture. Increasing in temperature were decreasing on rutting resistance for both SMA mix and SMAB mix. Nevertheless, SMAB have higher rutting resistance compare then SMA mix in each testing temperature. The slope of the relationship between the flow number vs temperature for SMAB mix, more horizontally compared to SMA mix, which indicates SMAB less sensitive against changes in temperature (higher PI).

The mineral aggregates constitute the rate of 90-95 % of the mixture weight and perform as skeleton and bearing member. Therefore, the physical properties of the mineral aggregates have noticeable effect on quality and characteristics of asphalt mixtures. One of the most important parameters in aggregates is grading, which is have influences on the rutting resistance. In this study, the effect of grading learned by making a variety of gradations, that is upper limit, lower limit and middle limit. The percentage of aggregates retain sieve number 4 and passes sieve number 200 were noted, as shown in Table 4 and Table 5. From figure 5, can be shown that increasing in percentage of filler (passes sieve no 200) were increasing the flow number value (rutting resistance).

Compacting the asphalt mixture is one of the most important factors in the flexible pavement construction, then have appropriate performance during his service life. Less compaction will cause the value of void in mix (VIM) larger. The effect of VIM on rutting resistance is presented in Figure 5. It shown that increasing in VIM were decreasing on rutting resistance or decreasing in the value of flow number.
Figure 3. Plot of flow number vs testing temperature

Figure 4. Plot of flow number vs percentage passing sieve no. 200

Figure 5. Plot of flow number vs void in mix
4 CONCLUSIONS

This study was conducted to investigate the effect of temperature, volumetric properties and mix grading on the rutting resistance (flow number properties) of the SMA mixture. The following conclusions can be drawn based on the interpretation of the test results.

- The addition of granular asbuton in the SMA mix has a significant effect on improving the rutting resistance, and granular asbuton can behave as a stabilizer and additional filler in the SMA mix.
- The flow number (rutting resistance) of SMA mixture observed in this study were decreases when the temperature and void in mix increases.
- On the same binder content, addition of the aggregate passes the sieve of no. 200 (filler) on a certain boundaries will increase the flow number.

5 ACKNOWLEDGEMENTS

The authors thanks the IRE pavement laboratory for his assistance in compiling and providing the test data.

REFERENCES

Bonaquist, R., (2012). Evaluation of Flow Number (Fn) as a Discriminating HMA Mixture Property, WHRP Report 12-01, Wisconsin Department of Transportation, Madison, WI


The Fatal and Serious Injury Risk of Motorcycle Collisions with Traffic Barriers

Hampton C. Gabler and Allison Daniello
Virginia Polytechnic Institute and State University
gabler@vt.edu

Abstract
Motorcyclists are vulnerable highway users, and are particularly at risk in collisions with traffic barriers, e.g. guardrail and wire rope barrier. Approximately one in eight motorcyclists who struck a guardrail are fatally injured – a fatality risk over 80 times higher than for car occupants involved in a collision with a guardrail. The objective of this study is to examine the emerging issue of fatal motorcycle collisions with traffic barriers, e.g. guardrail and concrete barrier. This issue is important internationally, but has profound implications for roadside design in regions such as Asia which place heavy reliance on motorcycles for transportation. In the U.S., motorcycle crashes have been found to be the leading source of fatalities in guardrail crashes. In 2004 for the first time, motorcycle riders suffered more fatalities than the passengers of cars or any other single vehicle type involved in a guardrail collision. In terms of fatalities per registered vehicle, motorcycle riders are dramatically overrepresented in number of fatalities resulting from guardrail impacts. In the U.S., motorcycles compose only 3% of the vehicle fleet, but account for 45% of all fatalities resulting from guardrail collisions. This paper will present the characteristics of serious injury and fatal motorcycle crashes into traffic barriers through examination of national data and in-depth accident investigations.

Introduction
Motorcycles have become increasingly popular and are a growing segment of the U.S. highway vehicle fleet. Unfortunately as the size of the U.S. motorcycle fleet has grown, the number of motorcycle fatalities has also grown. In the U.S. approximately 5000 motorcyclists died in traffic crashes in 2012, which accounted for 15% of all traffic deaths (NHTSA, 2012).

Motorcyclists are particularly at risk in collisions with traffic barriers, e.g. guardrail and wire rope barrier. Previous research in Europe, Australia, and the U.S. has found these crashes to be exceptionally dangerous for motorcyclists (Bambach et al, 2012; Berg et al, 2005; Candappa et al, 2005; Gabler, 2007; Gibson et al, 2000; Hell and Lob, 1993). In the U.S., one in eight motorcyclists who struck a guardrail were fatally injured – a fatality risk over 80 times higher than for car occupants involved in a collision with a guardrail (Gabler, 2007). This issue is important internationally, but has profound implications for roadside design in regions such as Asia which place heavy reliance on motorcycles for transportation.

Figure 1 presents the number of motorcyclists who were fatally injured in collisions with guardrail collisions from 1991-2012 in the U.S. To put this issue into context, the figure also presents the number of fatally injured occupants of cars, light trucks and all other highway vehicles. As shown, the number of fatalities of car occupants has steadily declined over this time period. This is the result of several factors including an increased seat belt usage rate over this time period, the widespread installation of airbags in the car and light truck fleet, and improved traffic barriers. In contrast to cars, the number of fatalities of motorcycles striking guardrail has steadily increased over this time period. In 2004, the U.S. reached a milestone. For the first time in history, motorcyclists exceeded the occupants of all other highway vehicles in the number of fatalities from guardrail collisions. In 2012, motorcyclists accounted for nearly half of all guardrail fatalities (45%) despite comprising only 3% of the U.S. highway fleet and less than 1% of the vehicles miles traveled.
Figure 1. U.S. Motorcycle-Guardrail Fatalities in comparison with other vehicle types (FARS 1991-2012)

The question for highway agencies is how this rising number of motorcycle-barrier fatalities can be reduced. Some motorcyclist advocacy groups in the U.S., Europe, and Australia have called for the removal of traffic barriers. However, this would be a serious disbenefit to other users. Traffic barriers, e.g., guardrail and cable barrier, are highly effective in protecting car and truck occupants who inadvertently depart the roadway. Improved roadside countermeasures are needed, but not by trading off car occupant safety for motorcyclist safety. Needed is a data-driven evaluation of motorcycle-barrier crash outcomes and injury mechanisms.

To investigate this issue, the Transportation Research Board of the U.S. National Academies of Science has sponsored National Cooperative Highway Research Program (NCHRP) Project 22-26, “Factors Related to Serious Injury and Fatal Motorcycle Crashes with Traffic Barriers”. This project has analyzed the influence of barrier design upon the outcomes of motorcycle-barrier collisions using both U.S. national and state accident databases. This paper will present and synthesize the findings of these studies.

**Objective**

This paper will present the characteristics of serious injury and fatal motorcycle crashes into traffic barriers through examination of U.S. national data and in-depth accident investigations. The longer term goal is provide the technical foundation for future barrier designs or design modifications which can provide protection for both motorcyclists while maintaining the safety of other road users in traffic barrier crashes.

**Approach**

The analyses which follow are based on the analysis of the three data sources: (1) the Fatality Analysis Reporting System (FARS), the National Automotive Sampling System – General Estimates System (NASS-GES), and state crash data. FARS is a census of all traffic-related fatalities that have occurred on public U.S. roads. The FARS dataset has been compiled every year since 1975, and is based on a fusion of information associated with fatal crashes.
including police accident reports, toxicology records and coroners’ reports. In 2012, the most recent dataset available at the time of this paper, FARS contained the records of over 33,000 fatally injured vehicle occupants, motorcyclists, bicyclists, and pedestrians killed in 2012 on public roads. In the analyses which follow, FARS will be used to compute the number of fatalities in roadside object collisions.

NASS-GES is a probability sample of approximately 60,000 crashes of the approximately 6 million police-reported crashes that occur in the U.S. each year. GES contains records of crashes of all severity levels ranging from property damage only to fatal crashes. The source for these records are police accident reports collected at sampling sites throughout the U.S. Each GES case has been assigned a weighing factor which when applied to GES tabulations allows the GES case counts to be escalated to obtain national estimates of crash and injury incidence. In the analyses which follow, GES will be used to estimate the U.S. exposure to roadside object collisions.

The analyses which follow also use state crash databases from three states: New Jersey, Texas, and North Carolina. These are databases of all police-reported crashes which occurred in the state each year. The injury severity varies from property damage only to fatality. State crash data has the advantage the crash locations have been geocoded which allow the type of barrier in each collision to be readily identified using products such as Google Earth or Google StreetView. The scope of the study will include motorcycle collisions into a wide range of traffic barriers including guardrail, cable barrier, concrete barrier, bridge rails, end-treatments, and crash cushions.

Results

Fatality Risk in Motorcycle-Barrier Crashes

Figure 2 presents the fatality risk of collisions with traffic barrier in comparison with other motorcycle collision modes (Daniello and Gabler, 2011a). The analysis was based on over 3600 fatal motorcycle crashes with roadside objects extracted from FARS 2004-2008. Exposure was estimated from motorcycle crashes of all types extracted from NASS/GES in order to estimate risks. The fatality risk of collisions with all objects has been normalized with the fatality risk of an overturn collision.

This analysis shows that the fatality risk of collisions with guardrail is over 7 times higher than an overturn crash. In comparison, collisions with concrete barrier have a fatality risk four times that of a ground collision – substantially
less than the risk of a guardrail collision. This is our first indication that barrier design is a likely factor in motorcycle crash injury outcomes. This figure also shows that there are other roadside objects which exceed the fatality risk of traffic barriers. Trees, utility poles, and highway signs have a fatality risk that ranges from 10-14 times higher than an overturn collision – double the fatality risk of a guardrail collisions. This point is important as frequently guardrail is used to shield motorists against collisions with these objects. Removing guardrail would expose motorcyclists to far worse collisions.

**Influence of Barrier Design upon Injury Risk**

The next step in the analysis was to determine the influence of barrier design on crash injury risk (Daniello and Gabler, 2011b). The objective was to determine whether some barriers are less dangerous than other designs in motorcycle-barrier crashes. Of particular interest was the crash performance of cable barrier systems which are relatively benign for car occupants but perceived by motorcyclists as a particular threat.

This study was based upon the analysis of state crash data from New Jersey, North Carolina, and Texas from 2004-2008. All three states had geocoded the location of each crash. The first step was to identify the barrier type at each crash scene. In state crash databases, the barrier type is typically coded only as metal or concrete. In many cases, the barrier type is missing or miscoded. In our analyses, we used Google Streetview and the crash geocoding to inspect photographs of the crash site, and code the actual barrier type.

<table>
<thead>
<tr>
<th>Barrier Type</th>
<th>New Jersey</th>
<th>North Carolina</th>
<th>Texas</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>W-beam</td>
<td>168</td>
<td>134</td>
<td>244</td>
<td>546</td>
</tr>
<tr>
<td>Concrete Barrier</td>
<td>87</td>
<td>23</td>
<td>248</td>
<td>358</td>
</tr>
<tr>
<td>Cable Barrier</td>
<td>0</td>
<td>15</td>
<td>32</td>
<td>47</td>
</tr>
<tr>
<td>Subtotal</td>
<td>255</td>
<td>172</td>
<td>524</td>
<td>951</td>
</tr>
<tr>
<td>No Barrier</td>
<td>21</td>
<td>10</td>
<td>347</td>
<td>378</td>
</tr>
<tr>
<td>Indeterminate</td>
<td>1</td>
<td>6</td>
<td>5</td>
<td>12</td>
</tr>
<tr>
<td>No Imagery</td>
<td>5</td>
<td>22</td>
<td>32</td>
<td>59</td>
</tr>
<tr>
<td>Total</td>
<td>282</td>
<td>210</td>
<td>908</td>
<td>1,400</td>
</tr>
</tbody>
</table>

As shown in Table 1, the records of 1400 motorcycle-barrier crashes were examined for this analysis. Of these 1400 crashes, the barrier could be identified in 951 cases and was separated into one of three groups: w-beam, concrete barrier, and cable barrier. In the remaining cases there was either no barrier at the geocoded crash site, no images of the crash site, or the barrier type could not be determined from the crash site images. These cases were excluded from the analysis.

Figure 3 presents the distribution of injury severity for each rider as a function of barrier type for helmeted riders. Injury severity in these databases was coded using the KABCO scale where K=killed or fatality, A=incapacitating injury, B=Moderate injury C=Complaint of Pain, and O=property damage only.
Severe injury in this analysis was defined to be the sum of K+A severities, i.e., riders with fatal or incapacitating injuries. For each barrier type, the odds of injury was computed, and then an odds ratio was computed to compare the likelihood of injury for two different barrier systems using the following equations:

\[
\text{Odds of Severe Injury} = \frac{\text{Probability (Severe Injury)}}{\text{Probability (Non-Severe Injury)}}
\]

\[
\text{Odds Ratio} = \frac{(\text{Odds of Severe Injury})_{\text{Barrier A}}}{(\text{Odds of Severe Injury})_{\text{Barrier B}}}
\]

Figure 4 presents the odds ratios of severe injury in w-beam vs. concrete, cable vs. concrete, and cable barrier vs. w-beam barriers. Also plotted are the 95th percentile confidence intervals. Confidence intervals which span an odds ratio of 1 indicate that there is no statistically significant difference in risk between the two barrier categories. This figure shows that w-beam carries 1.4 times higher odds of severe than does concrete barrier. This is consistent with the earlier analysis of fatal injury which showed that guardrail had a higher fatality risk than concrete barrier. Figure 4 also shows that there is no statistically significant difference in risk between cable and w-beam barrier. This could result from either of two reasons. Either the number of cable barrier cases (only 50) was too small to achieve statistical significance or there is legitimately no difference in the injury outcomes of these two barrier systems.
In-depth Crash investigations

Finally, the research project conducted a series of in-depth crash investigations to identify the actual barrier components and rider-barrier interactions which lead to serious rider injury. The previously presented studies determined the risk of serious and fatal injury by barrier type. However beyond barrier type, neither study was based on data sources sufficiently detailed to identify the specific injuries incurred by riders or the specific barrier components associated with these injuries. Both of these data elements would be needed to establish priorities for the design of barrier countermeasures.

Subjects for this study were enrolled after admission to the Wake Forest Medical Center Level 1 Trauma Center as a result of injuries sustained in a motorcycle-barrier collision. All subjects were seriously injured motorcycle riders and operators who consented to participate in the study. The data collection protocol was approved by Institutional Review Boards at both Wake Forest and Virginia Tech.

In-depth data collection for each case was comprised of three components. First, a crash investigator visited the crash site to collect and document scene evidence, measure the barrier characteristics, and photograph the crash scene. Second, the crash investigator conducted a post-crash motorcycle inspection in which the condition of the motorcycle and any damage to the motorcycle was documented with extensive photographs. Finally, all medical data on each subject was collected for the case. This included all applicable radiography and a complete list of injuries coded by severity using the Abbreviated Injury Scale (AIS). AIS rates on a six point scale. AIS 1 is a minor injury and AIS 6 is an unsurvivable injury.

To date, the study has enrolled 15 subjects of which 14 were operators and 1 was a passenger. Investigation of twelve (12) of these cases has been completed, with the remaining three cases currently underway. Figure 5 shows photographs of two of the cases.

Figure 6 presents the distribution of injuries from the first eight cases collected in the program. Although the number of cases is still limited at this point in the program, the plot presents some early trends. The most prevalent serious injuries (AIS 3-6) were to the thorax. All riders were helmeted and, likely as a consequence, serious head injuries were relatively rare. The most common non-serious injuries were to the upper and lower extremities.
Figure 5. Two cases from the NCHRP 22-26 Indepth Crash Investigations

![Two cases from the NCHRP 22-26 Indepth Crash Investigations](image)

Figure 6. Distribution of injuries from Indepth Crash Investigations (8 cases)

![Distribution of injuries from Indepth Crash Investigations](image)

For each serious injury, the barrier component or other impacting surface associated with each injury was coded. These injury sources were analyzed to identify priorities for countermeasure design. To date, the two major sources of injury have been identified. The first major source of injury was rider entanglement with the posts of the barrier.
system. The posts were associated with serious injuries in both w-beam and cable barrier collisions. The second source of injury occurred when a rider was partially ejected from the motorcycle onto the top of the barrier and then was dragged along the top of the barrier by the momentum of the motorcycle. Several riders experienced serious lacerations to the neck, chest, and abdomen from both the tops of the posts (both w-beam and cable), and from the sharp upper edge of the w-beam. In the two cases of cable barrier collisions in our dataset, there was no evidence of laceration from the cable barrier.

It should be emphasized that the results of the in-depth crash investigations presented here are based on a very limited sample size, and should not be construed as reaching statistical significance. Rather these cases studies should be used in conjunction with the two large scale studies on fatality and serious injury risk presented earlier, which did achieve statistical significance, to set countermeasure design priorities. The in-depth crash investigations are ongoing, and will continue to supplement these large scale studies with very detailed data on a smaller set of cases.

Conclusions
This paper has presented the findings of a study on serious injury motorcycle-barrier collisions in the U.S. In 2012, nearly half (45%) of all fatalities in guardrail collisions are motorcyclists. The study has shown that both fatality and serious injury risk are a function of traffic barrier design. The odds of severe injury in a w-beam guardrail collision were 1.4 times higher than those of a concrete barrier collision for a helmeted rider. Analyses of both large scale accident databases and in-depth crash investigations show no evidence that collisions with cable barrier is more dangerous than w-beam barrier. In both systems, the design priority appears to be the development and installation of countermeasures which can shield the tops of barriers and reduce post entanglement by riders.

Acknowledgments
The authors gratefully acknowledge the U.S. Transportation Research Board for sponsoring this research program under NCHRP Project 22-26.

References
A MASH Compliant Sign Mounting Designs for Placement on Concrete Median Barrier

By A. Y. Abu-Odeh, R. P. Bligh, and W. Odell

Authors:
Akram Y. Abu-Odeh (Corresponding Author)
Research Scientist
Texas A&M Transportation Institute
College Station, Texas 77843
O: (979) 862-3379
F: (979) 845-6107
Email: abu-odeh@tamu.edu

Roger P. Bligh
Research Engineer
Texas A&M Transportation Institute
College Station, Texas 77843
O: (979) 845-4377
F: (979) 845-6107
Email: rbligh@tamu.edu

Wade Odell, P.E.
Research Engineer
Safety & Operations
Structures & Hydraulics
Texas Department of Transportation
Research & Technology Implementation Office
P.O. Box 5080
Austin, Texas 78763-5080
O: (512) 416-4737
F: (512) 416-4734
Email: Wade.Odell@txdot.gov
Abstract

There is a growing need to place signs on median barrier as a way to relay vital traffic information to the traveling public. However, with the lack of MASH compliant sign mounting designs, state DOTs are limited to the Zone Of Intrusion (ZOI) guidelines to place such signs. The ZOI is derived from several NCHRP Report 350 tests and does not address the placement symmetry encountered in concrete median barriers (CMB) installation. In this paper, four different sign mounting designs are successfully crash tested according to MASH TL 3-11 that can be placed on top of a 32-inch or taller CMB. These mounting designs concepts include rigid, movable, hinged based and plastic yielding based mechanisms.

Keywords: Signs on medians, MASH, Median Barrier
INTRODUCTION

Concrete median barriers have been used throughout the nation as permanent and temporary barriers for providing separation of traffic. Typically, these barriers are tested and considered crashworthy through crash testing according to National Cooperative Highway Research Program (NCHRP) Report 350 or American Association of State Highway and Transportation Officials (AASHTO) Manual for Assessment of Safety Hardware (MASH) (1,2). Due to space restrictions, a sign or a light pole is placed on top of such barriers. However, when signs or light poles are mounted on top of barriers, the crashworthiness of the system is not necessarily guaranteed. There is very limited research on how a combination of device and barrier would perform if impacted by an errant vehicle. Therefore, there is a need to identify existing practices of placing hardware on top of median barriers, as well as defining the crashworthiness of such combinations.

BACKGROUND

Researchers at Midwest Roadside Safety Facility (MwRSF) developed the concept of Zone of Intrusions (ZOIs) as a guideline for the placement of attachments on top of or behind a barrier (3). They conducted a comprehensive review of full scale crash testing of bridge rail and median barriers to establish ZOIs for traffic barriers. A wide variety of traffic barrier classes including sloped-faced and vertical-faced concrete barriers were reviewed. ZOIs were identified for different NCHRP Report 350 test levels (1). Extent that a pickup or single truck intrudes over the top of barrier during an impact was the basis for establishing the ZOI. The maximum intrusions of any portion of a test vehicle beyond the top-front corner of the barrier were first considered as the definition of intrusion. For TL-3, barrier classes were combined into three groups based on the size of intrusion extent: (1) sloped face concrete barrier and steel tube rail on 6 inch curb or greater; (2) vertical face concrete barrier, combination of concrete and steel rail, all timber rail; and (3) steel tube rails not on a curb or on less than a 6 inch curb. ZOIs for TL-3 identified by Keller et al. are shown in Figure 1. Keller et al. recommended the placements of attachments outside the ZOI identified for each barrier class. Moreover, they recommended that the impact performance of an attachment and its placement that does not follow these suggested criteria can only be verified through the use of full-scale crash testing. More on the ZOI concept is presented in references (4) and (5).

Recently, a crash was performed on a TxDOT Type 2 portable concrete traffic barrier (PCTB) with a sign support assembly as per MASH test 3-11 (6). A crash test performed in 2001 on the modified TxDOT Type 2 PCTB with grid-slot connection and 1/4 inch thick steel straps. Sign support and sign mount connection was anchored on top of this modified concrete barrier in conjunction with the steel strap connections to three barrier joints. A 2270P (5000 lb) Dodge Ram 1500 pickup impacted the test article at a speed and angle of 63.4 mi/h and 24.6 degrees, respectively. The test successfully passed the safety evaluation criteria set forth in MASH test 3-11.
FIGURE 1 TL-3 Zone of intrusions for (a) sloped face concrete barrier and steel tube rail on curbs > 6 inches; (b) vertical face concrete barrier and combination concrete and steel rail; and (c) steel tube rail on curbs > 6 inches (3).

RESEARCH APPROACH

Researchers at TTI recently investigated the performance of a temporary concrete barrier with sign attachments mounted on top. The objective of the research was to develop a TxDOT standard for mounting traffic control signs and devices on concrete barrier.
The outcome of finite element simulations and engineering analyses of eight conceptual designs was that four concepts have the likelihood of passing MASH evaluation criteria. The simulation effort is not documented in this paper due to space restriction. The analyses were performed using up to 6 ft × 4 ft sign panel size. The concepts are:

- Schedule 80 post mounted rigidly on a spread tube.
- Hinge and sacrificial pin design.
- Sliding base and chute design.
- Slotted 10 BWG post (with 2-inch or 3-inch long slots).

Figure 2 shows these four concepts. Three concepts, the spread tube, the hinge with sacrificial pin, and the sliding base mounting were simulated using a 2.5-inch nominal size Schedule 80 post. The fourth concept, the slotted post, was simulated using a 2.5-inch nominal size 10 BWG post. The results of all detailed simulations indicated that these four concepts would pass MASH 3-11 test conditions within the accepted evaluation criteria.

**FIGURE 2 The four recommend sign mounting concepts.**

**CRASH TESTS RESULTS**

**Crash Test No. 466462-1 on the Spread Tube Sign Support System Mounted on CMB**

The barrier on which the spread tube sign support system was mounted contained and redirected the 2270P vehicle. The vehicle did not penetrate, underride, or override the installation. The sign support did not interfere with the ability of the barrier to contain and
redirect the vehicle. No movement in the barrier was observed. The sign post and the spread tube had insignificant damage as shown in Figure 3.

FIGURE 3 Spread tube concept before and after MASH TL 3-11 test.

The barrier on which the bracket and sacrificial pin sign support was mounted contained and redirected the 2270P vehicle. The vehicle did not penetrate, underride, or override the installation. The sign support did not interfere with the ability of the barrier to contain and redirect the vehicle. No movement of the barrier was observed. The sign post and the mounting bracket had insignificant damage as shown in Figure 4.

FIGURE 4 Hinge and sacrificial pin concept before and after MASH TL 3-11 test.
Crash Test No. 466462-3 on the Chute Channel Sign Support Mounted on CMB

The barrier on which the sliding base and chute channel sign support was mounted contained and redirected the 2270P vehicle. The vehicle did not penetrate, underride, or override the installation. The sign support did not interfere with the ability of the barrier to contain and redirect the vehicle. No movement of the barrier was noted but the post and the sliding base displaced along the chute as intended. The sign post and the mounting detailed had permanent deformation as shown in Figure 5.

![Image of sign support before and after](image1.png)

**FIGURE 5** The sliding base and chute concept before and after MASH TL 3-11 test.

Crash Test No. 466462-4 on the Slotted 10 BWG Sign Support on CMB

The barrier on which the slotted 10 BWG sign support was mounted contained and redirected the 2270P vehicle. The vehicle did not penetrate, underride, or override the
installation. The sign support did not interfere with the ability of the barrier to contain and redirect the vehicle. No movement of the barrier was noted. However, the post deformed and developed a plastic hinge around the slotted region as intended and the sign panel remained attached to the post but rested on the other side of the CMB as shown in Figure 6.

FIGURE 6 The slotted 10BWG sign concept after the test.

Tests Summaries

Figure 7 through Figure 10 present the test summary sheet for each test indicating that all of the four tests successfully passed the MASH evaluation criteria.
Test 3-11 on the spread tube sign support system on CMB.

<table>
<thead>
<tr>
<th>Test Article Deflections</th>
<th>Vehicle Stability</th>
<th>Exit Conditions</th>
<th>Soil Type and Condition</th>
<th>Test Vehicle</th>
<th>General Information</th>
</tr>
</thead>
<tbody>
<tr>
<td>Speed: 25 mph</td>
<td>Speed: 30 mph</td>
<td>Speed: 50 mph</td>
<td>Speed: 65 mph</td>
<td>2006 Dodge Ram 1500 Pickup</td>
<td>4900 lbs</td>
</tr>
<tr>
<td>Location/Station: 42.5</td>
<td>Location/Station: 42.5</td>
<td>Location/Station: 42.5</td>
<td>Location/Station: 42.5</td>
<td>Type: Driveway</td>
<td>4662 lbs</td>
</tr>
<tr>
<td>Cross Slope: 0%</td>
<td>Cross Slope: 0%</td>
<td>Cross Slope: 0%</td>
<td>Cross Slope: 0%</td>
<td>Street</td>
<td>0.0%</td>
</tr>
</tbody>
</table>

Vehicle Damage

- No damage
- Minor damage
- Major damage

Summary of results for FIGURE 7

- Vehicle: Test Vehicle
- Site: Street
- Time of Event: 0.000 s
- Speed: 65 mph

<table>
<thead>
<tr>
<th>Postimpact Recovery</th>
<th>Camera Conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.78 s</td>
<td>0.216 s</td>
</tr>
<tr>
<td>0.316 s</td>
<td>0.108 s</td>
</tr>
</tbody>
</table>

Test Stand 21 on the spread tube sign support system (TTT)
FIGURE 8 Summary of Results for MASH Test 3-11 on the Bridge and Safety Sign Support on CMB

Vehicle Damage

<table>
<thead>
<tr>
<th>Damage</th>
<th>Front</th>
<th>1.0</th>
<th>1.5</th>
<th>0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Front</td>
<td>0.28</td>
<td>0.01</td>
<td>0.06</td>
<td>0.05</td>
</tr>
</tbody>
</table>

Vehicle Function

<table>
<thead>
<tr>
<th>Function</th>
<th>Front</th>
<th>1.0</th>
<th>1.5</th>
<th>0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Front</td>
<td>0.28</td>
<td>0.01</td>
<td>0.06</td>
<td>0.05</td>
</tr>
</tbody>
</table>

Vehicle Stability

<table>
<thead>
<tr>
<th>Stability</th>
<th>Front</th>
<th>1.0</th>
<th>1.5</th>
<th>0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Front</td>
<td>0.28</td>
<td>0.01</td>
<td>0.06</td>
<td>0.05</td>
</tr>
</tbody>
</table>

Test Article Deflections

<table>
<thead>
<tr>
<th>Deflection</th>
<th>Front</th>
<th>1.0</th>
<th>1.5</th>
<th>0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Front</td>
<td>0.28</td>
<td>0.01</td>
<td>0.06</td>
<td>0.05</td>
</tr>
</tbody>
</table>

Occupant Risk Values

<table>
<thead>
<tr>
<th>Risk Value</th>
<th>Front</th>
<th>1.0</th>
<th>1.5</th>
<th>0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Front</td>
<td>0.28</td>
<td>0.01</td>
<td>0.06</td>
<td>0.05</td>
</tr>
</tbody>
</table>

Exit Conditions

<table>
<thead>
<tr>
<th>Exit Condition</th>
<th>Front</th>
<th>1.0</th>
<th>1.5</th>
<th>0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Front</td>
<td>0.28</td>
<td>0.01</td>
<td>0.06</td>
<td>0.05</td>
</tr>
</tbody>
</table>

General Information

<table>
<thead>
<tr>
<th>Information</th>
<th>Front</th>
<th>1.0</th>
<th>1.5</th>
<th>0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Front</td>
<td>0.28</td>
<td>0.01</td>
<td>0.06</td>
<td>0.05</td>
</tr>
</tbody>
</table>

Test Vehicle

<table>
<thead>
<tr>
<th>Vehicle Type</th>
<th>Front</th>
<th>1.0</th>
<th>1.5</th>
<th>0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Front</td>
<td>0.28</td>
<td>0.01</td>
<td>0.06</td>
<td>0.05</td>
</tr>
</tbody>
</table>

Soil Type and Condition

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Front</th>
<th>1.0</th>
<th>1.5</th>
<th>0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Front</td>
<td>0.28</td>
<td>0.01</td>
<td>0.06</td>
<td>0.05</td>
</tr>
</tbody>
</table>

Materials of Key Elements

<table>
<thead>
<tr>
<th>Element</th>
<th>Material</th>
<th>Front</th>
<th>1.0</th>
<th>1.5</th>
<th>0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Front</td>
<td>Steel</td>
<td>0.28</td>
<td>0.01</td>
<td>0.06</td>
<td>0.05</td>
</tr>
</tbody>
</table>

Sign Support

<table>
<thead>
<tr>
<th>Support</th>
<th>Type</th>
<th>Front</th>
<th>1.0</th>
<th>1.5</th>
<th>0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Front</td>
<td>Steel</td>
<td>0.28</td>
<td>0.01</td>
<td>0.06</td>
<td>0.05</td>
</tr>
</tbody>
</table>
FIGURE 9 Summary of results for MASH Test 3-11 on the chute channel sign support on CMB.

### Vehicle Damage
- Front: 3.5 G
- Left: 4.3 G
- Longitudinal: 4.5 G
- Right: 4.0 G

### Vehicle Position
- Maximum deflection: 0.92 ft
- Maximum deflection: 0.92 ft

### Vehicle Speed
- Speed: 0.110 s
- Speed: 0.000 s

### Test Vehicle
- Test Article: 2012-08-16
- Test Stand No.: Texas A&M Transportation Institute (TTI)
- Test Date: 11/11/11

### Test Conditions
- Impact Conditions: No damage
- Duty: 0.252 A
- Test Stand: MASH Test 3-11

### General Information
- Material of Key Elements: xx
- Side Fracture at 65 ft high
- Inlet: 20 ft long x 22 in. high
FIGURE 10 Summary of results for MASH Test 3-11 on the selected 10 BWG sign support on CMB.

### Vehicle Damage
- Vehicle type: \(10'\) Dodge Ram 1500 Pickup
- Vehicle damage: Front End, Right Wing, Right bumper, Right fender
- Vehicle condition: Destroyed

### Test Article Details
- Test Article: Reflective
- Diameter: \(0.288\) ft
- Height: \(2.33\) ft
- Weight: \(190.66\) lbs

### Impact Conditions
- Impact conditions:
  - Impact velocity: \(0.288\) ft/s
  - Impact force: \(6.84\) kips
- Vehicle Stability
- Post-impact Tension
- Roll Angle
- Front Angle
- Speed: \(0.49\) mph
- Height: \(1.87\) ft

### General Information
- Material of key elements: XX
- Installation: Vertical
- Manufacturer: Texas A&M Transportation Institute
- Test Article: \(0.288\) ft
- Test Speed: \(0.49\) mph
- Test Condition: Destroyed
- Test Support: Post-impact Tension
- Test Support on CMB
- Sign Support on CMB
- Soil Type and Condition: Barrier placed on concrete surface, dry

---

Abu-Abou, Bligh, and Odell
CONCLUSIONS

The following sign support designs were crash tested mounted on a concrete median barrier and were evaluated according to MASH guidelines for longitudinal barriers:

- Spread Tube Sign Support System.
- Hinge and Sacrificial Pin Sign Support System.
- Sliding Base and Chute Channel Sign Support System.
- Slotted 10 BWG Sign Support System.

None of the above sign support systems interfered with the ability of the concrete median barrier to contain and redirect the 2270P vehicles. As indicated earlier, each of the systems performed successfully according to the MASH criteria for longitudinal barriers.

New sign post on median barrier mounting designs have been developed and tested that allow placement of sign systems, with up to 4-ft $\times$ 6-ft sign, on permanent median or roadside barriers. These mounting designs were tested on the 32-inch tall NJ barrier because it is considered the most critical barrier profile. Hence, it is expected that these designs are applicable for F-shape and single slope profiles as long as they have a minimum of 32-inch height from the roadway surface.

The sliding base and chute design is the preferred design for implementation among the three listed above. The sign/post assembly would move along the chute once impacted by an errant pickup. The sign for the slotted 10 BWG post leaned down downstream and had 89.0 inches of maximum permanent deflection on the field side. So, the slotted 10BWG post concept will need enough clearance (i.e., wide shoulder width on each side of a CMB) to reduce potential interference with traffic. Practically, it should be used on roadside barriers or bridge rails. As for the hinge and sacrificial pin design, it did not activate in the crash test. Thus, it is not expected to activate for less severe impacts (nuisance hits). However, if activated, and the sign would lay down on the face of the barrier, then a clearance of 2 ft minimum is needed for the shoulder side on each side of the barrier.

ACKNOWLEDGEMENTS

This research project was conducted under a cooperative program between the Texas A&M Transportation Institute, the Texas Department of Transportation, and the Federal Highway Administration. The authors acknowledge and appreciate TxDOT guidance and assistance.

REFERENCES

5. Olson, D., Zone of Intrusion Concept Perspective from WA State DOT, Summer Meeting AFB20, Yountville, CA, 2010.
6. Williams, W.F., Menges, W.L., MASH Test 3-11 of TXDOT Portable Concrete Traffic Barriers (PCTB) with sign support Assembly, Publication: FHWA/TX-10/0-6143-1, Texas Transportation Institute, Texas A&m University System, College Station, TX, 2010.
Congratulations, Speed Limit, Congestion Thresholds, Mobility Performance Measures, Travel Time

ABSTRACT:
The question of “What is a proper congestion threshold?” has been debated among researchers and practitioners for a long time. One practice is the use of a speed limit-based value as the congestion threshold; another general practice is to allow the data to identify the speed in low volume conditions.

The research used the approach of a before-and-after quasi-experimental design. The study examined the changes in peak and off-peak average speed with three different speed limits on freeway sections in Houston, Texas over a two-year period. The travel time data was collected by a toll tag identification system or Automatic Vehicle Identification (AVI) system. The effect of confounding factors such as vehicle miles traveled (VMT) on travel time changes were also examined.

The research results show that the speed limit affects average speed in off-peak or free flow driving conditions but does not affect the speed distribution during congested driving conditions. However, the study also found that the off-peak speed may be noticeably higher than the speed limit, especially when the speed limit was lowered significantly for policy reasons. The two congestion threshold setting practices could show opposite effects on performance measures when speed limits are changed.
INTRODUCTION

In the United States, the question of “What is a proper congestion threshold?” has been debated among researchers and practitioners for a long time (1, 2). To date, there has not been a consensus on when congestion begins. Regardless of the differences, congestion has been monitored as part of the transportation system performance at both national and regional levels (3-7).

Two basic approaches exist for setting the congestion threshold. One approach uses the free-flow or unimpeded conditions as the congestion threshold (3, 4). With this approach of threshold setting, congestion measures all traffic delays beyond the free-flow or unimpeded conditions. The other approach uses the target or “acceptable” conditions as the congestion threshold (8, 9). The target or acceptable conditions are less ideal than the free-flow or unimpeded conditions. Within each approach there are more than one means of defining the free flow or the acceptable condition. For example, the Mobility Monitoring Program (4) used 85th percentile off-peak speeds as the congestion threshold for performance measures. For the target approach, one common practice uses a percentage of the speed limit as the congestion threshold. The 2011 Annual Congestion Report by Washington State Department of Transportation (WSDOT) (10) evaluated various system performance measures using average peak period travel speed below 85%, 75%, 70%, and 60% of the posted speed limit, respectively, as congestion thresholds. WSDOT believes that the maximum throughput speed, where the greatest number of vehicles can occupy the highway at the same time, usually occurs at between 70% and 85% of speed limit.

Both nationally and regionally, transportation agencies are struggling with defining the proper thresholds for evaluating transportation projects. Furthermore, many areas are considering either lowering speed limits for reducing energy consumption and mobile source emissions (11) or increasing speed limits for better mobility (12).

Limited research has been conducted on congestion threshold. Recent research (13) reveals that the rankings of freeway congestion levels hold steady across congestion thresholds and, therefore, the congestion threshold speed is not a concern for roadway improvement decisions.

This paper addresses a different congestion threshold issue -- the effect of changing speed limits (and the associated congestion threshold) on the performance measure values. In addition, the paper also examines the performance measure differences between the two congestion threshold approaches, namely, using the free flow speed or the speed limit as the indicator of the beginning of congestion. The specific objective is to examine the peak and off-peak average speeds with three different speed limit values over a period of 24 months. The goal is to provide evidence-based information about using the speed limit as the congestion threshold for calculating performance measures.

METHODOLOGY

An historical event was used for this study. Before May 2002, freeway speed limits in the Houston metropolitan area ranged from 60 mph to 70 mph. In May 2002, the speed limits for freeways were lowered to 55 mph area wide for environmental reasons. In September 2002, however, the speed limits were raised back to a speed limit that was 5 mph lower than the pre-May 2002 speed limit for the freeway sections where the pre-May 2002 speed limits were 65 mph or higher (14). In another words, the September 2002 speed limit was 60 mph for freeway with a pre-May 2002 speed limits of 65mph. June, July, and August of 2002 were the three full “during” months when the 55 mph speed limit was in effect. To rule out the seasonal effect, the same three months in 2001 and 2003 were selected as the “before” and “after” periods, respectively.

The specific steps involved to achieve the research objective were:

1) Exploring the trend of speed changes in the before, during and after months during peak and off-peak driving periods;
2) Comparing the difference between the posted speed changes and the average driving speed changes;
3) Identifying the confounding factors which may happen the same time as the speed limit changes and their roles in the speed changes; and
4) Estimating the effect of two threshold approaches on performance measures.

Source of Data

The archived traffic data from the Mobility Monitoring Program (MMP) were used for this research (4). The MMP is a data collection effort funded by the Federal Highway Administration (FHWA) to track and report traffic congestion and travel reliability using archived intelligent transportation system (ITS) data from many metropolitan regions. For the Houston metropolitan area, toll tag system data were used for MMP. The toll tag system collects the travel time directly when vehicles pass the toll tag readers; however, since not all vehicles have toll tags, the total vehicle volume traveling on the road section is not available from the toll tag system. The 5-minute standardized datasets from MMP which contain lane-by-lane speed were used for the study.
Qu and Lomax

Study Sites
One of the concerns in selecting study sites is that the inconsistency in law enforcement efforts may influence driving behaviors, which in turn, influences the travel time distribution of freeway segments. Both toll way and general public freeway sections were selected to explore the findings. The selected freeway segments are listed in Table 1.

<table>
<thead>
<tr>
<th>TABLE 1  Study Sites</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sections</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
<tr>
<td>5</td>
</tr>
<tr>
<td>6</td>
</tr>
<tr>
<td>7</td>
</tr>
<tr>
<td>8</td>
</tr>
<tr>
<td>9</td>
</tr>
</tbody>
</table>

Note: North bound direction of section 9 was omitted due to significant amount of missing data

Analysis Scenarios
The data aggregation uses three analysis periods of the same three calendar months.
- “high-speed” (before) period: June, July and August of 2001 (speed limit 60 to 70mph)
- “55 mph” (during) period: June, July and August of 2002 (speed limit 55mph for all sections)
- “middle-speed” (after) period: June, July and August of 2003 (speed limit 60 to 65mph)

Both off-peak, peak periods and peak hour are examined for the effect of speed limit change on travel time distribution. The designations of the three time periods are,
- Off-peak period: Six hours of early morning (12:00 am to 6:00 am) and five hours of late night (7:00 pm to 12:00 am)
- Peak periods: Three hours (6:00 am to 9:00 am) of morning peak and three hours (4:00 pm to 7:00 pm) of evening peak
- Peak hours: One hour (7:00 am to 8:00 am) of morning peak hours and one hour (5:00 pm to 6:00 pm) of evening peak hours

Data Aggregation
The standardized 5-minute speed data set from MMP was used for this study. The toll tag data aggregation process was applied to aggregate the data. The detailed aggregation procedures can be found in a separate research report (15).

RESULTS

Daily Speed Profile
The three-month weekday aggregated speeds for each 5-minute time slice in a day were plotted for the nine study sections and the three analysis periods(Figure 1). For the off-peak period, all study sections showed the lowest speed in 2002 (the “during” period) when the 55mph speed limit was in effect area wide. However, the off-peak speeds were noticeably higher than 55 mph for all sections in 2002. The off-peak speeds for 2001 (“before” period) seem the highest and the 2003 (the “after” period) speeds were in between the “before” and “during” periods.

Average Speed of Off-Peak Period, Peak Period, and Peak Hour
The 5-minute speeds were further aggregated to the off-peak, peak period, and peak hour. The speed aggregation was done separately for the toll and non-toll freeway sections due to the possibility of inconsistent law enforcement efforts which may results in different driving behavior. Figure 2 shows the average speeds of different section groups for the three analysis periods and time of day periods.
FIGURE 1  Daily Speed Profiles of the Three Analysis Periods for the Nine Study Sections.

FIGURE 2  Average Speeds of Off-peak, Peak Period and Peak Hour for Section Groups.

Identifying Confounding Factors
A before-and-after quasi-experimental design can be described as the approach for the study (16). The reason that the approach is not an experimental design is that the researchers could not control the population group, location, timing, or manner in which the speed limit was lowered. Many other factors/events that happened at the same time as the speed limit change may have also contributed to the observed speed changes and provided potential alternative explanations for the findings. However, this problem can be overcome if the likelihood of the rival events can be discounted.
Qu and Lomax

**Off-Peak Period**

For the off-peak period, the trend of average speeds followed the trend of the speed limits; the “before” period is the highest, the “during” period is the lowest, and the “after” period is in between (see Figure 2). The Analysis of Variance (ANOVA) test was performed for all 5-minute speeds in the off-peak period. The result indicated that the average speeds for the three analysis periods are significantly different from each other regardless of section groups. However, the average speeds for the off-peak periods were significantly higher than the speed limits in effect. For 2002, the average off-peak speed about 7 to 8 mph higher than the area wide 55 mph speed limit (see Figure 2).

Confounding factors, such as severe weather events, incidents, and road construction, were examined for the three months in 2002. None of the confounding factors were believed to be abnormal compared to the same period in the other two years. Therefore, the speed limit change was the factor that affected the vehicle speed change in the off-peak period. However, the magnitude of the effect may not be as significant as the changes in speed limit values.

**Peak Period and Peak Hour**

It is believed that during forced-flow conditions, volume is the determining factor for speed distribution. Since vehicles can no longer operate at a free flow speed during the peak period and peak hour, the posted speed limit does not significantly affect the vehicle speed under forced-flow conditions. This assumption can be explained by the speed and volume relationship of the traffic flow theory (17). Figure 3 illustrated the approximate areas where the average speeds of the off-peak, peak period, peak hour and the worst 15 minutes of the day fall in the speed flow diagram. According to the traffic flow theory, speed decreases as the flow level increases up to the maximum flow under heavy traffic conditions. Therefore, average speed and volume for the peak period and peak hour would be expected to have the opposite trend for the three analysis periods.

Both the ANOVA and Tukey’s Studentized Range tests were performed for all 5-minute speeds in the peak period and peak hour. The result indicated that the average speed for the “before” period is significantly higher than the “during” and “after” periods and the difference of average speeds between “during” and “after” periods were not significant.

![Speed and Volume Diagram](image)

**FIGURE 3** Freeway Speed and Volume Diagram for Time Periods.

As introduced in the data source section, the toll tag data does not have volume; the Annual Average Daily Traffic (AADT) data were used to represent the traffic volume change during the peak period and peak hour. Table 2 lists the aggregated AADT and the percentage change of AADT for the six non-toll freeway sections (toll section AADT data is not available). The 2001 AADT was the lowest among the three years for all non-toll sections. The combined non-toll sections showed the same level of AADT in 2002 and 2003.
TABLE 2  
AADT for the Non-Toll Sections in the Analysis Periods

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>I-10 Katy Between Barker Cypress and Sam Houston</td>
<td>1 &amp; 2</td>
<td>175,029 (4%)</td>
<td>167,757 (14%)</td>
<td>147,320</td>
</tr>
<tr>
<td>I-10 Katy Between Sam Houston and I-610</td>
<td>3 &amp; 4</td>
<td>184,920 (-5%)</td>
<td>195,135 (19%)</td>
<td>164,492</td>
</tr>
<tr>
<td>US 59 Between Wilcrest and I-610</td>
<td>5 &amp; 6</td>
<td>281,750 (0%)</td>
<td>281,551 (5%)</td>
<td>268,530</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>641,699 (0%)</td>
<td>644,443 (11%)</td>
<td>580,343</td>
</tr>
</tbody>
</table>

Source: Texas Department of Transportation (TxDOT)

Figure 4 depicts the AADT trend and the observed and expected speed trend for the three analysis periods. The fact that trend of speed followed the reverse direction of the AADT proves that volume is the determining factor for speed during forced-flow conditions.

FIGURE 4  
Observed and Expected Peak Period Speed Trend for the Analysis Periods.

Impact on Performance Measures
The fact that peak period speed was not influenced by the speed limit change makes the speed limit a good candidate for being used as the congestion threshold in performance evaluation. However, the evidence that the actual free flow speed was higher than the speed limit indicates that congestion may have already started when vehicles are driving at the speed limit (if “congestion” is defined beginning when speed falls below free-flow speed).

Two popular performance measures were selected to illustrate the effect of using the speed limit versus the free flow speed as the congestion threshold.

1. Total Delay
The total delay is used to measure congestion magnitude. Delay was defined as the additional vehicle travel time that is greater than the free flow vehicle travel time.

\[
\text{Total Segment Delay (vehicle - hours)} = \left( \frac{\text{Segment length (miles)}}{\text{Average speed (mph)}} - \frac{\text{Segment length (miles)}}{\text{Congestion threshold speed (mph)}} \right) \times \frac{\text{Vehicle Volume (vehicles)}}{}
\]
2. Travel Time Index (TTI)

TTI is used to measure congestion intensity. It is the ratio of time spent in traffic during peak traffic times to free flow traffic times. It was calculated in this paper as the ratio of congestion threshold speed to average travel speed.

\[
TTI = \frac{\text{Congestion threshold speed (mph)}}{\text{Average travel speed (mph)}}
\]

The section 6 (8.3 miles) data was selected to illustrate the performance evaluation. It was assumed that 30% of the AADT occurred in the three hour morning peak period. Table 3 illustrates the performance measure comparisons between an approach based on 1) the speed limit (Approach 1) and 2) an approach based on the measured off-peak speed (Approach 2) for the three analysis years each of which has a different speed limit.

| Table 3 Effect of Two Congestion Threshold Approaches on Performance Measures |
|------------------|----------------|--------|--------|
|                  | 2001        | 2002    | 2003    |
| Speed Limit      | 65.0        | 55.0    | 60.0    |
| Measured Off-Peak Speed (mph) | 66.2        | 63.7    | 66.5    |
| Average Peak Period Speed (mph) | 52.7        | 47.9    | 51.5    |
| Travel Time Index | Approach 1  | 1.23    | 1.04    | 1.17    |
|                  | Approach 2  | 1.26    | 1.33    | 1.29    |
| % increase from  | Approach 1 to| 13%    | 725%    | 70%    |
| Approach 1 to    | Approach 2* |        |        |        |
| Delay (Vehicle Hours) | Approach 1 | 2,404   | 1,892   | 1,932   |
|                  | Approach 2  | 2,590   | 3,635   | 3,076   |
| % increase from  | Approach 1 to| 8%     | 92%     | 59%     |
| Approach 1 to    | Approach 2* |        |        |        |

*Note: when calculating the changes on Travel Time Index, the number before the decimal point should be ignored.

The results show that performance measures using the measured off-peak speed approach are higher than that using the speed limit approach. This is because the measured off-peak speeds are higher than the speed limit speeds. The results also show that the two approaches to threshold setting would have determined very different conclusions from the 2002 speed limit change. If the speed limit was used as the congestion threshold, year 2002 was the least congested both in terms of intensity and magnitude. However, if the measured off-peak speed was used as the congestion threshold, year 2002 was the most congested both in terms of intensity and magnitude. The different effects are due to the artificially low standard (congestion threshold) brought on by the speed limit change.

CONCLUSIONS

The issue of the appropriate speed threshold is embedded in the congestion performance measures. If the speed limit is used as congestion threshold, the performance measures would change accordingly when the speed limit changes. The results show that 1) the speed limit is one of the factors affecting vehicle speed in the off-peak or free flow driving condition period and 2) the speed limit has no effect on speed distribution during congested driving conditions. However, the study also found that the off-peak speed may be noticeably higher than the speed limit, especially when the speed limit was lowered significantly for policy reasons. The two congestion threshold setting practices, namely the speed limit and the measured off-peak speed, could show opposite effects on performance measures when the speed limit is changed. Therefore, it may be necessary to investigate the free flow speed if the speed limit-based approach is used for performance measures when evaluating the effect of a speed limit change.
REFERENCES


Characterization of Pedestrian Fatalities in Urban Arterial Corridor in Puerto Rico

**AUTHOR**

Benjamín COLUCCI
- Professor, Abertis Chair and Spokesperson of Decade of Action for Road Safety, Commonwealth of Puerto Rico
- University of Puerto Rico at Mayagüez
- Puerto Rico

**CO-AUTHOR(S)**

Dafne VALLE-JAVIER
- Graduate Student
- University of Puerto Rico at Mayagüez
- Puerto Rico

**E-MAIL**

benjamin.colucci1@upr.edu

**KEYWORDS:**

pedestrian crash rate, pedestrian fatalities, pedestrian facilities, urban corridor, Puerto Rico

**ABSTRACT:**

Pedestrian crash related fatalities are a social and governmental concern worldwide. The National Highway Traffic Safety Administration ascribed to the United States Department of Transportation has placed the Commonwealth of Puerto Rico as the location with the highest pedestrian fatality rate in the United States with 2.71 fatalities per 100,000 population, and an average of 31% of all traffic related fatalities on the Island primarily occur on urban corridors.

An evaluation of pedestrian fatalities was conducted on an urban arterial corridor located on the Western Region of Puerto Rico. Hazardous segments based upon pedestrian crash data were determined. A database developed using police crash records showed that urban corridors in the Municipality of Mayagüez has the highest frequency of pedestrian fatal crashes of the region with 42%. The highest frequency of pedestrian crashes occurred in July and during the peak hours of 6:00 PM to 9:00 PM. Cross tabulation showed that the highest pedestrian fatalities occurred between kms. 154-156 on PR-2, an eight lane, urban arterial where public residential housing, elementary and high schools, and government services are located. Observational studies performed on this corridor indicated that the lack of pedestrian refuge island, and an insufficient pedestrian signal phase in the intersections are contributory factors of pedestrian crashes in these locations.
Characterization of Pedestrian Fatalities in an Urban Arterial Corridor in Puerto Rico

Benjamín Colucci-Ríos PhD, PE, PTOE, FITE, PAE, JD and Dafne Valle-Javier, ME

1University of Puerto Rico at Mayagüez, USA
Email for correspondence: benjamin.colucci1@upr.edu

2 University of Puerto Rico at Mayagüez, USA
Email for correspondence: dafne.valle@upr.edu

INTRODUCTION
Pedestrian fatalities are a major concern to local and federal government officials. Recent statistics published by the National Highway Traffic Safety Administration (NHTSA) ascribed to the United States Department of Transportation has placed our island with the highest pedestrian fatality rates in the United States with a total of 2.71 fatalities per 100,000 population (NHTSA 2012). In Puerto Rico (PR), pedestrian fatalities represent 31% of all traffic fatalities, which is almost 300% as compared to the national figure of 11% in the United States. Based upon this alarming pedestrian fatality statistic, a research study was conducted as part of the Dwight D. Eisenhower Fellowship Program for Hispanic Serving Institutions funded by the Federal Highway Administration with data obtained from the Bureau of Highway Patrol of the Puerto Rico Police Department (PRPD), and the data of the Puerto Rico Traffic Safety Commission (PRTSC). This pilot study concentrated on the Western Region, including its surrounding 9 municipalities and focusing on the PR-2 Urban Corridor which is part of the National Highway System.

The goal of this study was to identify hazardous pedestrian locations based upon current geometric and operational characteristics that complement the randomness associated with pedestrians, and recommend proper countermeasures to help reduce the pedestrian fatalities in the area. Pedestrian fatalities affects emotionally the family and friends of the victim, and creates a comprehensive crash cost of $4 million USD per fatality crash, depending upon the productivity years that the victim cannot produce to society, and the mental damages and suffering which affects the family’s future relations at both home and work (AASHTO 2010).

OBJECTIVES
The objective of this study is to characterize pedestrian crash data of the Western Region, which is located on the southwest corner of the island, and to identify potential hazardous sites based on observation and pedestrian crash data. The study period covers the period from 2007 to 2013.

METHODOLOGY DESCRIPTION
Figure 1 summarizes the methodology used in this research study. Initially, a literature review regarding pedestrians including articles, laws, regulations and publications was performed. Data collection regarding crash fatalities was conducted by reviewing crash reports from the Puerto Rico Police Department (PRPD). The data collected included the 9 municipalities comprising the Western Region of the Commonwealth of Puerto Rico. Other data collected included crash fatality data from the Puerto Rico Traffic Safety Commission (PRTSC 2012) and the Fatality Analysis Reporting System (FARS), and the information regarding pedestrian crashes in the United States published by the National Highway Traffic Safety Administration (NHTSA 2012).

Statistical analysis included tabulations, histograms and cross tabulations. Observational studies were performed to evaluate the pedestrian facilities and pedestrian behavior along the PR-2 Urban Corridor. Countermeasures were recommended using Highway Safety Manual (HSM) methodology with Crash Modification Factors.
LITERATURE REVIEW

The “Traffic Safety Facts 2010” published by the National Center for Statistics and Analysis of the NHTSA summarized facts involving pedestrian fatalities in the United States (NHTSA 2012). In the United States, on average a pedestrian is killed every two hours in traffic crashes. The NHTSA reported an increase of 4% of pedestrian fatalities in the year 2010 in comparison with the year 2009, with a total of 4,280 fatalities. In 2010, approximately 67% of the victims were male, almost 30% of the crashes occurred between 8:00 PM to 11:59 PM and approximately 50% of the fatalities occurred during the weekends.

A research study conducted in Puerto Rico characterized pedestrian crashes on urban highways of the Island (Alicea 2004). The Accident Analysis Office of the Department of Transportation and Public Works (DTPW) and FARS provided the databases for this investigation. The investigation concluded that male pedestrians are three times more susceptible than woman, that young pedestrians of 20 years or younger are more susceptible than other age groups, and the peak hours of pedestrian crashes are from 6:00 PM to 10:00 PM.

Public perception regarding the condition of pedestrian infrastructures on the island inferred that in many intersections the pedestrians traffic signals are vandalized, the marking at the crosswalks are deficient and many sidewalks had poor pavement condition which are hazardous to ordinary pedestrians (Cortés 2013).

ANALYSIS

Table 1 summarizes the data collected in the PRTSC website and the information regarding pedestrian crashes in the United States published by the NHTSA. During the last decade, pedestrian fatalities represent an average of 31% of all traffic fatalities, which is almost 300% as compared to the national figure of 11% in the United States. This statistic should raise awareness to government agencies in the process of prioritizing pedestrian needs in the planning and design stages.
Table 1. Pedestrian Fatalities in the Commonwealth of Puerto Rico and the US: Study Period 2002-2011

<table>
<thead>
<tr>
<th>Year</th>
<th>Commonwealth of Puerto Rico (PRTSC 2012)</th>
<th>United States (NHTSA 2012)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total Fatalities</td>
<td>Pedestrian Fatalities</td>
</tr>
<tr>
<td>2002</td>
<td>519</td>
<td>177</td>
</tr>
<tr>
<td>2003</td>
<td>495</td>
<td>150</td>
</tr>
<tr>
<td>2004</td>
<td>495</td>
<td>162</td>
</tr>
<tr>
<td>2005</td>
<td>457</td>
<td>134</td>
</tr>
<tr>
<td>2006</td>
<td>508</td>
<td>140</td>
</tr>
<tr>
<td>2007</td>
<td>452</td>
<td>145</td>
</tr>
<tr>
<td>2008</td>
<td>406</td>
<td>130</td>
</tr>
<tr>
<td>2009</td>
<td>365</td>
<td>109</td>
</tr>
<tr>
<td>2010</td>
<td>340</td>
<td>101</td>
</tr>
<tr>
<td>2011</td>
<td>361</td>
<td>111</td>
</tr>
<tr>
<td>Total</td>
<td>4,398</td>
<td>1,359</td>
</tr>
</tbody>
</table>

* In 2010, the reported pedestrian fatality rate for Puerto Rico is 2.71 per 100,000 population (NHTSA 2012)

The NHTSA reported the pedestrian crash fatalities per 100,000 populations for all the territories of the United States using pedestrian crash fatalities for the year 2010. Puerto Rico has the highest index with approximately 2.71, followed by Florida with an index of 2.58, Delaware with an index of 2.45, Arizona with an index of 2.28, the District of Columbia with 2.15 and South Carolina and Hawaii with 1.94.

The Traffic Division of the Bureau of Highway Patrol of the PRPD is the entity in charge of monitoring the principal roads in the different police regions of the island. The Police Academy has specialized police personnel that goes to the crash scenes, and gathers the information using a standardized police crash report.

Table 2 summarizes the crash fatalities data within the region from the period covering the years of 2007 through February of 2013 and the classification of the fatalities. A total of 170 fatalities were reported in the region during the study period.

Table 2. Classification of Crash Fatalities in the Western Region of Puerto Rico: Study Period 2007-2013

<table>
<thead>
<tr>
<th>Year</th>
<th>Classification of the Fatalities</th>
<th>Total Fatalities</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Driver</td>
<td>Passenger</td>
</tr>
<tr>
<td>2007</td>
<td>12</td>
<td>6</td>
</tr>
<tr>
<td>2008</td>
<td>8</td>
<td>5</td>
</tr>
<tr>
<td>2009</td>
<td>11</td>
<td>4</td>
</tr>
<tr>
<td>2010</td>
<td>9</td>
<td>3</td>
</tr>
<tr>
<td>2011</td>
<td>12</td>
<td>4</td>
</tr>
<tr>
<td>2012</td>
<td>8</td>
<td>4</td>
</tr>
<tr>
<td>2013*</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Totals</td>
<td>61</td>
<td>27</td>
</tr>
</tbody>
</table>

*Only two months were accounted for the year 2013

Table 2 shows the classification of crash fatalities in the Western Region of Puerto Rico for the period of 2007-2013. Based on these data, the highest number of fatal crashes was reported in 2009 with a total of 34 fatalities. The total percentages of the classification of traffic fatalities were 36% of those killed in a crash were drivers, 29% were pedestrians, 16% were passengers, 12% were motorcyclists, 5% were cyclists and 2% were classified as others. The others classification represents a passenger of a motorcycle and two horse riders that were using the road. The 29% percent of the pedestrians killed in this region is consistent with the pattern of the total pedestrians killed in the island, which is approximately 30%.

The Traffic Division of the Bureau of Highway Patrol has reported 50 pedestrian fatalities in the last 6 years. Table 3 has a summary of the municipalities of the region, the population of the municipalities reported by the 2010 census, the total pedestrian crash fatalities for each municipality reported for that period, the pedestrian fatality rate per 10,000 population, and the percentage of the total pedestrian fatalities. The Municipality of Mayagüez had the highest percentage of pedestrian fatalities with a total 42%.
Table 3. Pedestrian Crash Fatalities and Fatality Rate by Municipality: Western Region of Puerto Rico

<table>
<thead>
<tr>
<th>Municipality</th>
<th>Population</th>
<th>Total Pedestrian Fatalities</th>
<th>Pedestrian Fatalities per 10,000 Population</th>
<th>Percentage (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Añasco</td>
<td>29,261</td>
<td>4</td>
<td>1.37</td>
<td>8</td>
</tr>
<tr>
<td>Cabo Rojo</td>
<td>50,197</td>
<td>14</td>
<td>2.79</td>
<td>28</td>
</tr>
<tr>
<td>Hormigueros</td>
<td>17,250</td>
<td>3</td>
<td>1.74</td>
<td>6</td>
</tr>
<tr>
<td>Lajas</td>
<td>25,753</td>
<td>3</td>
<td>1.16</td>
<td>6</td>
</tr>
<tr>
<td>Las Marías</td>
<td>9,881</td>
<td>1</td>
<td>1.01</td>
<td>2</td>
</tr>
<tr>
<td>Mayagüez</td>
<td>89,080</td>
<td>21</td>
<td>2.36</td>
<td>42</td>
</tr>
<tr>
<td>Sabana Grande</td>
<td>25,265</td>
<td>2</td>
<td>0.79</td>
<td>4</td>
</tr>
<tr>
<td>San Germán</td>
<td>35,527</td>
<td>2</td>
<td>0.56</td>
<td>4</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>282,214</strong></td>
<td><strong>50</strong></td>
<td>N/A</td>
<td><strong>100%</strong></td>
</tr>
</tbody>
</table>

Figure 2 shows the information related to the gender and group age distribution of male pedestrians. In terms of gender, 68% of the victims were male and 32% were female. Almost 48% of the male fatalities were above 56 years of age.

Figure 2. Distribution of Pedestrian Fatal Crashes According to Gender and Age Group

A pertinent aspect regarding pedestrian crash fatalities is the frequency distribution in terms of the month and day of the week of the crash incident. Figure 3 summarizes the information related to month and day of the week of the occurrence of the pedestrian crash. In terms of months, the highest frequency corresponded to the month of July with 16%, followed by November with 12%. The southwest corner of the island is a popular tourist destination mainly because of the great beaches and protected wildlife areas. The month of July is the peak of the tourism season in the region, and that may be the rational explanation for the incidence increase of pedestrian crash fatalities during that particular month.

In terms of the day of the week, Friday has the highest frequency reported with 30%. Of the pedestrian crashes that were reported on Friday, 60% were after 7 PM or during nighttime, and 40% were before 7 PM. During the weekend, which includes Friday night, Saturday and Sunday, the frequency was almost 46% of the pedestrian fatal crashes.
Figure 3. Pedestrian Fatal Crashes According to Month and Day of the Week of Crash Occurrence

Figure 4 shows the distribution of number of pedestrian fatalities per hour of the day. Specifically, it shows that the top frequency observed was from 7:00 to 8:00 PM, with 16% of the total pedestrian fatalities. There were two peak hours, one during early morning (12:00 AM to 4:00 AM), with 24% of the total pedestrian fatalities, and the other during the period of 5:00 PM to 9:00 PM, with 38% of the total pedestrian fatalities. Importantly, 78% of the fatalities occurred during nighttime.

Figure 5 shows the distribution of pedestrian maneuvers prior to motor vehicle - pedestrian crashes for the 6 year period. The pedestrian maneuvers with highest frequency were: crossing outside an intersection or midsegment with 32%, followed by walking on the roadway against traffic with 24%, and walking on the roadway in favor of traffic with 15%. The majority of the rural roads of the western region lack pedestrian facilities such as adequate sidewalks and illumination which can endanger pedestrians.
Figure 5. Pedestrian Maneuvers Prior to Motor Vehicle - Pedestrian Crashes

Figure 6 shows the distribution of probable causes of motor vehicle – pedestrian crashes obtained from Police Crash Reports and narrative descriptions prepared by the investigative officer. The highest frequency, 43%, of probable crash causes was attributed to inadequate or absence of illumination on site.

Figure 6. Probable Causes of Motor Vehicle - Pedestrian Crashes

The pedestrian crash analysis showed that the PR-2 Urban Corridor in the City of Mayagüez has the highest incidence of pedestrian fatalities with an approximate 26% of the total fatalities of the Western Region. The PR-2 Urban Corridor is classified as a high speed, urban arterial that crosses the city from north to south. The segment that was evaluated has an AADT with a range between 50,000 to 80,000 vehicles per day. The urban corridor has frontage roads in several segments which provide access to traffic generators such as commercial, industrial, governmental and housing developments. Unfortunately, the frontage roads are not continuous thus, creating gaps in which schools and commercial generators are contiguous to the main highway combined with the lack of continuous sidewalks for pedestrian use.

In order to identify segments in the corridor with high frequency of pedestrian fatalities, cross tabulations and crash rates analysis were performed. The PR-2 Urban Corridor in the Municipality of Mayagüez is ranked among the top ten most hazardous highways in terms of motor vehicle-pedestrians crashes on the Island.
The results of the cross tabulation analysis showed that the highest incidence of pedestrian crashes on the PR-2 Urban Corridor occurred between kilometers 154 to 156, specifically between the Mayagüez bypass to Duscombe Street Intersection, with a frequency of 62% of the total pedestrian crashes on that particular road. This segment has a high density of pedestrians due in part to a large quantity of pedestrian generators such as schools, government agencies, commercial and residential areas.

In order to perform an accurate crash analysis of the PR-2 Urban Corridor it is pertinent to identify the roadway segments and intersections. To calculate the crash rates that take into account the exposure data of the roadway segment in consideration the following formula from the Highway Safety Manual (HSM) is used (AASHTO 2010):

\[ R = \frac{C \times 10,000,000}{V \times 365 \times N \times L} \]  

(1)

Where:
- \( R \) = crash rates for road segments expressed as crashes per 100 million vehicle-kilometers traveled
- \( C \) = number of pedestrian fatal crashes in the study period
- \( V \) = Average Annual Daily Traffic (AADT)
- \( N \) = study period, years
- \( L \) = segment length, kilometers

The crash data were obtained from the Traffic Division of the Region of Mayagüez of the PRPD. The information related to the AADT was obtained from the Puerto Rico DTPW. The AADT used was from the year 2008. To calculate the segment pedestrian crash rates, the PR-2 Urban Corridor was divided based upon the AADT’s available between intersections of the corridor. Table 4 shows the location of the segments and the calculated crash rates for all the road segments of the PR-2 Urban Corridor in the Municipality of Mayagüez.

Table 4. Calculated Crash Rates for Segments on the PR-2 Urban Corridor: Study Period 2007-2013

<table>
<thead>
<tr>
<th>Segment</th>
<th>Number of Crashes</th>
<th>Length (km)</th>
<th>AADT (veh/day)</th>
<th>Crash Rates</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Salinas to Carolina</td>
<td>1</td>
<td>1.2</td>
<td>66,650</td>
<td>0.46</td>
</tr>
<tr>
<td>Street</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2. Carolina to Centro</td>
<td>1</td>
<td>0.5</td>
<td>77,200</td>
<td>1.18</td>
</tr>
<tr>
<td>México Entrance</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3. Centro Médico to</td>
<td>1</td>
<td>1.1</td>
<td>77,200</td>
<td>0.54</td>
</tr>
<tr>
<td>Duscombe Avenue</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4. Duscombe to</td>
<td>2</td>
<td>0.8</td>
<td>68,700</td>
<td>1.64</td>
</tr>
<tr>
<td>Nenadich Street</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5. Nenadich Street to</td>
<td>3</td>
<td>0.31</td>
<td>59,800</td>
<td>7.30</td>
</tr>
<tr>
<td>the Viaduct</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6. Viaduct to Llorenz</td>
<td>3</td>
<td>1</td>
<td>51,500</td>
<td>2.66</td>
</tr>
<tr>
<td>Torres Avenue</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7. Llorenz Torres to</td>
<td>1</td>
<td>0.7</td>
<td>54,600</td>
<td>1.19</td>
</tr>
<tr>
<td>Chardón Street</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8. Chardón to PR-306</td>
<td>1</td>
<td>0.8</td>
<td>51,600</td>
<td>1.12</td>
</tr>
</tbody>
</table>

Table 4 shows the variables and the calculated crash rates for the PR-2 Urban Corridor in the Municipality of Mayagüez. The segment between the Nenadich Street and the entrance to the viaduct had the highest crash rates with 7.39 crashes per 100 million vehicles – kilometers traveled.
Observational studies were performed of the intersections along the PR-2 Urban Corridor. A checklist was developed based upon AASHTO’s Guide for the Planning, Design and Operation of Pedestrian Facilities (AASHTO 2004). The elements evaluated were the sidewalks, crosswalks, pedestrian refuge islands, and the pedestrian traffic signals for the urban corridor. A total of 7% of the sidewalks did not comply with the 1.52 meters minimum width requirement, and 14% of the crosswalks did not comply with the 1.83 meters minimum width requirement. A total of 43% of the intersections did not provide pedestrian refuge island, and a total of 7% of the pedestrian traffic signals were not operating properly. Another area of concern is the pedestrian signal timing. It is pertinent to emphasize that the lack of adequate storage for pedestrians, combined with the pedestrian walking interval of 12 seconds is not adequate for crossing 6 lanes plus two turning lanes, that has an approximately length of 30.5 meters.

DISCUSSION

The Commonwealth of Puerto Rico has the highest pedestrian fatality rate in the United States with 2.71 fatalities per 100,000 population. Furthermore, during the last decade the pedestrian fatalities, on average, correspond to 31% of all traffic related fatalities on the island, primarily concentrated in urban corridors with traffic generators on both sides and discontinuities in the built infrastructure in terms of frontage roads and sidewalks.

During the study period of 2007-2013, a total of 170 pedestrians were killed in motor vehicle-pedestrian crashes in the Western Region of Puerto Rico. The 29% of the pedestrians killed in the region are compatible with the pattern of the total pedestrian fatalities for the island (31%). The City of Mayagüez has the highest frequency of pedestrian fatal crashes with a total of 42%.

The characterization of the pedestrian victims of the region involved an analysis of the gender, age distribution, month, day and time of the incident, the maneuvers of the victim before the incident, and the probable causes. In terms of the gender of the pedestrian crash victims, 68% of the victims in the region were male, which is similar to United States’ statistics of 66%. Almost 48% of the male victims were above 56 years old. This statistic reflects that older people have a higher risk of being involved in a motor vehicle - pedestrian crash due to their mobility and reaction time, which makes them vulnerable while crossing a street. The month of July had the highest frequency of pedestrian fatalities in the region with a total of 16%. The region is a tourist destination because of the spectacular beaches and wildlife reserves. The peak of tourism is during the month of July, this rational explanation can be the contributing factor for such statistic. During the weekend which includes Friday night, Saturday and Sunday, the frequency was almost 46% of the total pedestrian fatal crashes, which is similar to the statistic reported by NHTSA of the frequency of almost 50% of occurrence of pedestrian fatalities during weekends in the United States. The peak hours for pedestrian fatalities were during nighttime between 6 PM to 9 PM with approximately 34% of the total pedestrian fatalities. The pedestrian maneuvers with highest crash frequency were crossing outside an intersection with 32%, followed by walking against traffic with 24%, and walking with the flow of traffic with 15%. In terms of probable causes for motor vehicle – pedestrian crashes, inadequate illumination was the probable cause for 43% of the cases.

A major achievement was to identify a hazardous segment within the PR-2 Urban Corridor. Cross tabulations and crash rate analysis determined that the segment between Nenadich Street and the viaduct had the highest incidence of pedestrian fatalities on the Corridor. The PR-2 Urban Corridor has a concentration of public residential housing, schools and government services that generates large quantity of pedestrians. In terms of traffic control devices, the observational studies performed showed a lack of a pedestrian refuge island and only 12 seconds for pedestrians to clear the crosswalk. During the night, the illumination provided by light poles were not uniform in the vicinity of the intersections evaluated.

CONCLUSIONS AND RECOMMENDATIONS

The PR-2 Urban Corridor in the City of Mayagüez has the highest incidence of pedestrian crashes with approximately 26% of the total pedestrian fatalities of the region. Even though the pedestrian crashes are located in segments, there is an overlap with the intersections contiguous to the segment. Lack of enforcement of pedestrian crossing in crosswalks is a contributory factor for this high pedestrian fatality rate. Short term improvements for the corridor include the optimization of the traffic lights without increasing traffic delays. According to the HSM Crash Modification Factors, implementing this countermeasure can reduce the motor vehicle – pedestrian crashes by 50%.
The segment within the PR-2 Urban Corridor between the Nenadich Street and the viaduct had the highest incidence of pedestrian fatalities. Recommended countermeasures for the intersections along this segment are the construction of a pedestrian refuge island, improvement of the illumination system, and increasing the pedestrian walking interval phase to better accommodate the necessities of the users.

An inventory of the condition of the pedestrian facilities, and an analysis of potential countermeasures for each intersection on the PR-2 Urban Corridor was performed and was submitted to Puerto Rico Highway and Transportation Authority for evaluation and implementation. This assessment could potentially assist in the improvement of pedestrian facility conditions, and reducing pedestrian fatalities along the corridor. Awareness campaigns are also recommended to move toward a culture of safety with zero pedestrian fatalities in Puerto Rico.

ACKNOWLEDGEMENTS

Special acknowledgement to the PRPD personnel and to the Dwight D. Eisenhower Fellowship Program for Hispanic Serving Institutions sponsored by the Federal Highway Administration (FHWA) for the awarded fellowship.

REFERENCES


ABSTRACT:
Longitudinal rumble strips have proven to be an effective treatment on paved shoulders in preventing roadway departure crashes on two lane rural roads.

In 2009, the Government of Puerto Rico started the implementation of longitudinal shoulder rumble strips on freeways. In 2010, the Highway Safety Manual (HSM) was published by AASHTO providing tools for decision making and to estimate how effective a countermeasure or set of countermeasures will be in reducing crashes at a specific location. Crash Modification Factors (CMF) are used to quantify the effect of a particular treatment on expected crash frequency adjusting the estimate of crash frequency from a base condition defined by a Safety Performance Function (SPF) to the specific conditions present at a site. In the first edition of 2010 HSM, CMF’s for freeway applications using non-continuous longitudinal rumble strips on shoulders were not included.

This paper documents the process of developing CMF for non-continuous rumble strips on urban and rural freeway segments, in rolling to mountainous topography, and the first phase of the development of SPF’s. The study area is PR-52, a 108.3 km toll freeway facility which is part of the United States National Highway System (NHS), and includes level, rolling and mountainous areas with an ADT ranging from 26,700 vpd to as high as 166,500 vpd.
INTRODUCTION

The road network of the Commonwealth of Puerto Rico consists of 26,866 centerline kms of which 26,720 are paved and 146 unpaved. In 2012, the Federal Highway Administration (FHWA) of the US Department of Transportation (USDOT) reported 29,915 millions of vehicle-kms traveled on the island highway network of which 39.6% corresponded to the National Highway System (NHS). The interstate system, which includes rural and urban freeways with toll facilities, generated 29.6% of all vehicle-kms traveled (FHWA 2013).

During the last decade, approximately 4,397 motor-vehicle crash fatalities were reported in the island of which approximately 25% are roadway departure crashes (TSC 2013). A preliminary analysis of roadway departure crashes, using a 5 year moving average, resulted in 134 fatalities. Furthermore, an alarming 1.96 fatalities/100 million vehicle-miles traveled (MVMT) were reported as compared with 1.13 fatalities/100 MVMT in the continental US (FARS 2013).

Based on these facts, the Puerto Rico Department of Transportation and Public Works (DTPW) concentrated their efforts to improve safety of the highway network by approving the Puerto Rico Strategic Highway Safety Plan (SHSP 2014). In the SHSP, roadway departure crashes have been identified as one of the areas of emphasis, based upon significant contributing causes of traffic fatalities and serious injuries in the island. The revision of roadside safety engineering policies and standards, and education of innovative roadway departure countermeasures, such as non-continuous longitudinal rumble strips, are listed as strategies that have the potential of reducing roadway departures during the next 5 years on high speed freeways within the NHS interstate system.

Even though non-continuous longitudinal rumble strips are perceived as a cost effective countermeasure to reduce roadway departure crashes, the initial edition of the 2010 Highway Safety Manual (HSM) published by AASHTO does not include Safety Performance Functions (SPF’s) and Crash Modification Factors (CMF’s) for freeway applications. In 2009, Puerto Rico Highway and Transportation Authority (PRHTA) started the implementation of non-continuous longitudinal rumble strips on freeway shoulders.

This paper will focus on the development of SPF’s for freeway segments, combining historical crash data with the Highway Performance Monitoring System Database of the Commonwealth of Puerto Rico for a 2 year and 3 year study period.

OBJECTIVES

The objective of this research is three fold:

- Provide an overview of rumble strips to address roadside departure crashes in high speed freeway facilities.
- Develop SPF using exposure and historical crash data from the Puerto Rico HPMS.
- Perform a sensitivity analysis to evaluate the effect of the independent variables, AADT, and segment length on the expected number of crashes for the study period.

LITERATURE REVIEW: RUMBLE STRIPS

Rumble strips are a safety feature or treatment on a paved roadway, capable of alerting drivers that their vehicle is leaving the travel lane. In the United States, this special treatment has proven to be effective (FHWA 2013). Rumble strips can be either permanent or provisional. Examples of permanent installment of rumble strips are
along the centerline of a two way roadway or on the shoulder of a roadway. In the case of provisional rumble strips, they are commonly used to alert vehicles of changes in the roadway. Temporary transverse rumble strips in work zones are commonly used in high speed rural freeway facilities to assist in speed reduction of vehicles approaching the work zone. Essentially transverse rumble strip are used to alert drivers of a potential change or hazards in the roadway. Figure 1 illustrates longitudinal and transverse rumble strip applications.

Figure 1. Longitudinal and Transverse Rumble Strips (Source: http://safety.fhwa.dot.gov/)

**DESIGN GUIDELINES FOR RUMBLE STRIPS**

The PRHTA issued a Design Guidelines No. 409 for the installment of rumble strips in the island highway system (PRHTA, 2012). Table 1 provides a comparison between the FHWA Technical Advisory 5040.39, the National Cooperative Highway Research Program (NCHRP) Report 641, and the PRHTA Design Directive No. 409.

Table 1. Local and National Specifications for Shoulder Rumble Strips

<table>
<thead>
<tr>
<th>REQUIREMENTS</th>
<th>FHWA (TA 5040.39)</th>
<th>NCHRP (641) Most Common Values</th>
<th>PRHTA (DD#409)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-Minimum Shoulder Width (feet)</td>
<td>4</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>B-Lateral Clearance (inches)</td>
<td>9</td>
<td>12</td>
<td>12</td>
</tr>
<tr>
<td>C-Rumble Strips Width (inches)</td>
<td>7</td>
<td>7</td>
<td>7</td>
</tr>
<tr>
<td>D-Rumble Strips Length (inches)</td>
<td>16</td>
<td>16</td>
<td>16 to 18</td>
</tr>
<tr>
<td>E-Center to Center Spacing (inches)</td>
<td>Not specified</td>
<td>12</td>
<td>12</td>
</tr>
<tr>
<td>Rumble Strips Depth (inches)</td>
<td>1/2</td>
<td>½</td>
<td>1/2 to 5/8</td>
</tr>
<tr>
<td>Bicycle Gap (feet)</td>
<td>10 to 12</td>
<td>10 to 12</td>
<td>6 to 12</td>
</tr>
<tr>
<td>Minimum Posted Speed (mph)</td>
<td>50</td>
<td>45 to 50</td>
<td>Not specified</td>
</tr>
</tbody>
</table>
RESEARCH METHODOLOGY

The development of SPF’s are part of a large scale research project that will ultimately develop CMF’s for non-continuous shoulder rumble strips on Puerto Rico freeways. The development of SPF’s correspond to Phase A of the flowchart shown in Figure 2.

Figure 2. Research Methodology

Two databases were used in Phase A to develop SPF’s, namely historical crash and HPMS databases. They provided traffic (AADT, %Trucks), geometric (horizontal curve, grades), and operational (speed limit) characteristics, as well as crash data for a study period covering from 2006 to 2012.

During the summer of 2009, PRHTA performed a pilot project that consisted of the installation of milled-in, non-continuous shoulder rumble strips along the PR-52 freeway. The study area is highway PR-52, a 108.3 kilometers long toll freeway facility that is part of the National Highway System, which originates from the north in San Juan, Capital of Puerto Rico, crossing the central mountain range, and ends in the City of Ponce. Both cities have shipping ports that receive and deliver commercial freight. Its AADT ranges from 165,800 vpd in the origin, an urban area with level to rolling terrain, to a minimum of 18,600 vpd in a rural mountainous region. The maximum speed limit is 65 miles per hour and has approximately 8% of heavy trucks.

Figure 3 shows the warning sign with the rumble strips phrase in Spanish within the text plate, “huella en el paseo”, alerting drivers of the presence of longitudinal shoulder rumble strips, followed by a representative longitudinal shoulder rumble strip on NHS PR-52 freeway, and the recommended dimensions of the
longitudinal shoulder rumble strip. In our research project, the non-continuous longitudinal shoulder rumble strip installed on this freeway are 5.7 feet (1.74 m) long, 1 feet (0.30 m) wide and 10.8 feet (3.29 m) gap between the strips.

![Image](Image.png)

Figure 3: (a) Rumble Strips Sign, (b) Rumble Strips on NHS PR-52 Freeway; (c) Sketch of the Gap Dimension of the NHS PR-52 Non-continuous Rumble Strips

DEVELOPMENT OF SAFETY PERFORMANCE FUNCTIONS (SPF’s)

SPF is a statistical model that is used to predict crashes in the future, at a particular location such as a road, segment or intersection. The first step of the Empirical Bayes Method (EB) was the development of a SPF for both total crashes and injuries. It includes the creation of a reference group that is a collection of untreated segments that have similar characteristics as the treated segments. Table 2 shows the distribution of total crashes for the freeway study segments of the NHS, namely PR-22 and PR-52.

<table>
<thead>
<tr>
<th>Road Name</th>
<th>2006</th>
<th>2007</th>
<th>2008</th>
<th>2009</th>
<th>Total Crashes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (km)</td>
<td>Total Crashes</td>
<td>Crashes /km-yr</td>
<td>Total Crashes</td>
<td>Crashes /km-yr</td>
<td>Total Crashes</td>
</tr>
<tr>
<td>PR-22</td>
<td>0.0-83.7</td>
<td>738</td>
<td>8.82</td>
<td>816</td>
<td>9.75</td>
</tr>
<tr>
<td>0.0-108.3</td>
<td>970</td>
<td>8.96</td>
<td>933</td>
<td>8.61</td>
<td>831</td>
</tr>
</tbody>
</table>

The segment selection for the reference group was a combination of treated segments in the NHS PR-52 and untreated segments of the NHS PR-22 with similar characteristics. The segmentation is based upon the segmentation use in the Highway Performance Monitoring System (HPMS) Database which defines segments for each road in Puerto Rico based upon the Annual Average Daily Traffic (AADT). Table 3 presents the principal characteristics of the segments selected for the reference group of NHS toll freeways using 2009 as the base year.
Table 3. Characteristics of the Segments Selected for the Reference Group

<table>
<thead>
<tr>
<th>Characteristics of the Segments of the Reference Group</th>
<th>NHS PR-52</th>
<th>NHS PR-22</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Lanes</td>
<td>4 to 6 lanes</td>
<td>4 to 6 lanes</td>
</tr>
<tr>
<td>Lane Width</td>
<td>12 feet (3.65 m)</td>
<td>12 feet (3.65 m)</td>
</tr>
<tr>
<td>Average Segments AADT’s (vehicles/day)</td>
<td>70,677</td>
<td>77,438</td>
</tr>
<tr>
<td>Average Crashes for Segments (per year)</td>
<td>30</td>
<td>23</td>
</tr>
</tbody>
</table>

The second major task was the data cleaning process in which incomplete records were eliminated from the database. The data cleaning process was performed on all of the segments for both freeways including the reference group. A total of 491 crash records were eliminated because they lacked the exact location of the crash or had errors related to the exact kilometer location.

The calibration of the preliminary SPF was completed using a methodology proposed by Hauer in which he suggests that an SPF can be built by adding the variables in the model equation, one at a time. If the modeler reports every SPF gradually obtained, practitioners then can use the model for which they have the data available (Hauer 2014). Hauer suggests to start the modeling process with a segment length as a simple model equation, and then add the rest of the variables. The development of the SPF’s were obtained by using a curve fitting model in combination of a function in Microsoft Excel called the “Solver Parameter Tool” which can solve the parameters of practically any function that would better fit the model.

The SPF’s were developed assuming a Negative Binomial Distribution (NBD). In the past, researchers used Poisson Distribution but recently had shown that the Negative Binomial Distribution offers better fitted models than the Poisson Distribution. A pertinent parameter for the development of the EB method is the negative binomial dispersion parameter ($\Phi$) obtained from this regression.

The first preliminary SPF’s were performed by fitting a power function. On the first trial, the segment length (kms) was used as the independent variable. The first SPF is represented by equation (1), where $E(\mu)$ is defined as expected crashes, $X_1$ is the segment length (kms) and $\beta_0$ and $\beta_1$ are regression parameters.

$$E(\mu) = \beta_0 \times X_1^{\beta_1} \quad (1)$$

Table 4. Results of the Models with Segment Length for 2 to 3 Year Periods

<table>
<thead>
<tr>
<th>Severity Type</th>
<th>Expected Crash Frequency Model (2 Year Period)</th>
<th>Expected Crash Frequency Model (3 Year Period)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>regression parameters</td>
<td>over-dispersion parameter</td>
</tr>
<tr>
<td></td>
<td>$\beta_0$</td>
<td>$\beta_1$</td>
</tr>
<tr>
<td>Total Crashes</td>
<td>22.245</td>
<td>0.737</td>
</tr>
<tr>
<td>Crashes with Injuries</td>
<td>21.899</td>
<td>0.737</td>
</tr>
</tbody>
</table>

Table 4 provides the values of the parameters ($\beta$), over-dispersion parameter ($\Phi$) and the value of the Pearson Index obtained by the Microsoft Excel Solver Parameter Tool. The Pearson Function Index can range from -1 to 1 and reflects the relationship between two data sets. On the initial analysis, the Pearson Function Index value was low for all of the models with a SPF based upon segment length. To better improve the SPF, an additional variable, the AADT for each segment, was added. The second SPF is represented by equation (2), where $E(\mu)$ is defined as expected crashes, $X_1$ is the segment length (kms), $X_2$ as the AADT (vehicles/day) and $\beta_0$, $\beta_1$ and $\beta_2$ are regression parameters.
\[ E(\mu) = \beta_0 * X_1^{\beta_1} * X_2^{\beta_2} \] (2)

**Table 5. Results of the Models with Segment Length and AADT’s for 2 to 3 Year Periods**

<table>
<thead>
<tr>
<th>Severity Type</th>
<th>Expected Crash Frequency Model (2 Year Period)</th>
<th>Expected Crash Frequency Model (3 Year Period)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>regression parameters</td>
<td>over-dispersion parameter</td>
</tr>
<tr>
<td></td>
<td>( \beta_0 )</td>
<td>( \beta_1 )</td>
</tr>
<tr>
<td>Total Crashes</td>
<td>0.00042</td>
<td>0.847</td>
</tr>
<tr>
<td>Crashes with Injuries</td>
<td>0.00037</td>
<td>0.855</td>
</tr>
</tbody>
</table>

Table 5 provides the values of the parameters (\( \beta \)), over-dispersion parameter (\( \Phi \)), and the value of the Pearson Function Index obtained by Excel for the SPF’s that included the segment length and the average AADT’s variables. The Pearson Function Index gets closer to 1 and reflects that there is a better relationship between the two data sets (observed vs. fitted values).

In order to evaluate the effect of the independent variables AADT and segment length on the expected crashes, a sensitivity analysis was performed. The study period for this analysis was 2 years. Figure 5 shows the results of the sensitivity analysis with segment lengths varying from 0.5 kms to 5.0 kms, and AADT from 25,000 vpd to 75,000 vpd, which is representative of the traffic exposure in the NHS rural segments, where rumble strips were installed. Based on this analysis, the SPF generates a good fit in terms of increasing the number of expected crashes with an increase in traffic exposure (AADT) and segment length.

**Figure 5. Sensitivity analysis of the effect of AADT (\( X_2 \)), and segment length (\( X_1 \)) on the expected number of annual crashes on freeway segments of varying lengths with non-continuous rumble strips 2 year model**

**CONCLUSIONS**

This paper focused on the development of SPF’s for freeway segment reference groups, combining historical crash data with the HPMS database provided by the Puerto Rico Department of Transportation and Public Works, and performing a sensitivity analysis to evaluate the effects of the independent variables, segment length and AADT, on the expected number of crashes.
The results of Phase A of the research study showed that in general:

The Microsoft Excel Solver Parameter Tool can be useful in the preliminary stages of developing SPF’s by adding independent variables in a step wise fashion. The Solver Parameter Tool is a very flexible and powerful tool since it allows the researcher to calibrate different mathematical functions.

In terms of the first model, with a SPF based upon segment length, it was noted that the Pearson Function Index value was low, therefore, the estimate of the expected number of crashes with only segment length as an independent variable was not reliable.

In terms of the second model, with a SPF based upon segment length and AADT, the Pearson Index gets closer to 1, and reflects that there is a better relationship between two data sets (observed vs. fitted values) to estimate the expected number of crashes in the study period.

In terms of the sensitivity analysis, the model is a powerful tool to assess the fitness of the SPF with two independent variables. Based on the sensitivity analysis, the SPF generates a good fit in terms of increasing the number of expected crashes with an increase in traffic exposure (AADT) and segment length.

Ongoing studies will apply the Empirical Bayes Method to generate CMF’s associated with non-continuous shoulder rumble strips for freeway application.

ACKNOWLEDGEMENTS

Special thanks to the Road Safety Audit Division of the Puerto Rico Transportation and Highway Authority for providing the databases for the development of the SPF, and to the personnel of the Puerto Rico Transportation Technology Transfer Center at the Mayagüez Campus for their assistance in the data analysis.

REFERENCES


PAPER TITLE
(90 Characters Max)
IMPRESS THE REGIONAL ACCESSIBILITY THROUGH ROAD NETWORK DEVELOPMENT IN THE BORDER REGION OF INDONESIA

TRACK

AUTHOR
(Capitalize Family Name)
Gede Budi SUPRAYOGA
Head of Section
Institute of Road Engineering
Indonesia

CO-AUTHOR(S)
(Capitalize Family Name)
POSITION
ORGANIZATION
COUNTRY

E-MAIL
(for correspondence)
gede.budi@pusjatan.pu.go.id

KEYWORDS:
Include up to 5 keywords
border region, accessibility, road network, strategies, regional development, Trans-ASEAN Highway

ABSTRACT:
The abstract should be written in English, readily understandable to most readers and may contain up to maximum of 200 words. Authors are invited to use Times New Roman Size 10 and the full type width of the page (single column).

Road network construction in the cross-border region of Indonesia is considered important to support the economic development, to ensure the national security, and to integrate with the regional agenda, such as Trans-ASEAN and Trans-Asian Highways agreement. The urge to construct more roads and to improve the recent road condition is undeniable considering the lack of accessibility of the region and the gap in economic development with other regions. The purposes of the paper are to identify related factors that promote the need of road infrastructure development in the cross border region of Indonesia, as well as to identify the current road network profile of the region. Firstly, the paper elaborates the paradigm shift of the cross border development policies in Indonesia and their implications to road infrastructure development. From a case study, the paper captures the condition of the road infrastructure at the Padang-Gajingan Besar area which is a part of the Kaimantan – Sarawak State (Malaysia) border region. The study indicates that road network development in the area needs to be intensified, especially to serve accessibility for the people living in the area. Finally, some key strategies of road network development are proposed to support the national policies on cross border development and to harmonize with the Trans-ASEAN Highway agenda.
IMPROVE THE REGIONAL ACCESSIBILITY THROUGH ROAD NETWORK DEVELOPMENT IN THE BORDER REGION OF INDONESIA

Gede Budi Suprayoga
Institute of Road Engineering, Agency for Research and Development, Ministry of Public Works of Indonesia
gede.budi@pusjatan.pu.go.id

Road network construction in the cross-border region of Indonesia is considered important to support the economic development, to ensure the national security, and to integrate with the regional agenda, such as Trans-ASEAN and Trans-Asian Highways agreement. The urge to construct more roads and to improve the recent road condition is undeniable considering the lack of accessibility of the region and the gap in economic development with other regions. The purposes of the paper are to identify related factors that promote the need of road infrastructure development in the cross border region of Indonesia, as well as to identify the current road network profile of the region. Firstly, the paper elaborates the paradigm shift of the cross border development policies in Indonesia and their implications to road infrastructure development. From a case study, the paper captures the condition of the road infrastructure at the Paloh-Sajingan Besar area which is a part of the Kalimantan – Sarawak State (Malaysia) border region. The study indicates that road development in the area needs to be expanded, especially to serve accessibility for the people living in the area. Finally, some key strategies of road network development are proposed to support the national policies on cross border development and to harmonize with the Trans-ASEAN Highway agenda.

Key words: border region, accessibility, road network, strategies, Trans-ASEAN Highway

1. INTRODUCTION

For a long period, the cross-border regions of Indonesia have been neglected regarding their economic development. The regions were lagged behind in term of economic and infrastructure development indicators compared to other parts of the country (Husnadi 2006, Nugroho 2013). The regions usually do not have adequate access to economic institution that can lend credit for business incubations. Inadequate investment in physical infrastructure also puts the region in the stagnant situation and isolation. The regions are not attractive unless government opens the access of the region. Some people lived in the border tend to travel to the nearest countries to find better jobs and public services because of the lack of intra-region’s accessibility to access them in Indonesia. For example, people in Sambas, Kalimantan Barat, have to travel more than 90 km or about 10 hours to find the nearest public hospital in Sambas while Kuching, the capital state of Sarawak (Malaysia), provides the service which is only less than 4 hours in travel distance supported with a good quality road condition.

Started in 2005, the government of Indonesia has put some efforts to boost the regional development of the border region through infrastructure construction and local economic stimulation schemes. There is also a change in perspective of the regions from “inward looking” into “outward looking”. “Inward looking” means that the economic potencies of the border region was only needed to support the development that happened in Java or other more developed parts of the country. In addition, the development of the region was only seen as its national security aspect, while neglecting its economic prospects. This perspective directed that the national policies that the border region was a supporting spatial and economic system in the country’s development. Otherwise, the outward looking paradigm sees the border region as an important gate for economic activities and trade as well as a main locus for national development (Government of Indonesia 2008, Hansen & Annovazzi 2008, Javi 2011). The paradigm also influences policies in infrastructure development to support prosperity, while at the same time to maintain the national security agenda.

This paper elaborated the paradigm shift of the cross border development policies in Indonesia and their implications to road infrastructure development. This paper focuses on a specific border region of Indonesia called Paloh-Sajingan Besar which is located between Sambas (Indonesia) and Sarawak (Malaysia). From a case study of the region, the paper captures the condition of the road infrastructure at the Paloh-Sajingan Besar area. The study indicates that road development in the area needs to be expanded, especially to serve accessibility for the people living in the area and to interact intensely with border regions. Some key strategies of road network development are proposed to support the national policies on cross border development and to harmonize with the Trans-ASEAN Highway agenda.
2. POLICIES ON THE BORDER REGIONAL DEVELOPMENT IN INDONESIA

Komornicki (2005) indicated that a country’s border has been initially established to configure an imaginary line that politically distinct the two countries. This phase has been called as the first transformation of the region as a military function. The initial phase is then followed by expansion of socioeconomic functions. Based on Komornicki’s observation in European countries, the economic and social functions takes over the whole transformation manifested in customs services as well as in traffic restrictions between the countries. The final development is indicated by intensified trade activities in among regions in the border area. A cross border region can also be viewed as barriers regarding human and goods traffics or can be called as non-tariff barriers (Button 2001). Commonly, the area shows geographical obstacles that hamper goods and people to be transported easily, such as mountains and forest areas.

In Indonesia, a change in military to economic function had been promoted in the spatial plan concept of the Kasaba (Kalimantan-Sarawak-Sabah) border region five years before the National Spatial Plan (RTRWN) was signed in 2008. The concept endorsed the regional development through spatial and regional development at the level of Economic Development Area (KPE) which consists of districts at the country’s borderline. The area was developed based on competitive products produced in those areas which will be traded with the neighbour country. Spaces for development were allocated for small and medium industries as well as infrastructure and economic facility. Susantono (2009) mentioned that the role of this area is as a centre for the promotion, investment, commercial, and office, and settlement.

The RTRWN mentioned that there were three land-based and six sea-based border regions in Indonesia. Sambas- Sarawak, is part of the Heart of Borneo border area and called as the National Strategic Area (KSN) which was established to support the national security and defence, showing the character of the region with its military function. The RTRWN indicated that the KSN is also dedicated as a regulator for cross-border movement and for trade through inspection posts. The function endorses other functions including monitoring for illegal trading and anticipating diseases transmitted from foreign animals.

Recently, the border regional development emphasized on three approaches: prosperity, security and national defence, and environmental sustainability (Government of Indonesia, 2008; Sambas, 2011a). The prosperity approach is an effort to develop economic and trade activities to improve the welfare of the people. The concept of security (the security approach) looks at the border region as an area adjacent to other countries that need oversight for its strategic defence and security values, including keeping the investment climate. The environmental approach provides a perspective that in order to sustain the environment and to minimize the impacts; the regional development must be controlled as well.

3. THE ROLE OF ROAD INFRASTRUCTURE IN THE BORDER REGION OF SAMBAS

For an area with a spread distribution of settlements, such as the border region, the transportation demand is commonly low compared to other areas in the vicinity. This prompted transportation services served monopolistic or dominant role of government driven (Anderson, 2001). The active role of the government, both central and local, is required to provide transport services in the region. The road networks are designed based on the functions of activity centres in the settlement system.

There are two competing concepts in road network development in the border region of Indonesia. As a region with a defence function, road network should allow a quick response to security threats and efficient military movement. The concept that is offered is "the belt defence" that allows thorough patrol and surveillance as well as a swift military mobilisation to encounter attacks from another country (Soesetyo 2009). In this concept illustrated in Figure 1, the roads network link with the borderline and develop into a grid type connecting the administrative centres of the smallest (sub-districts) until district centres. The first defensive line is a border security zone or a buffer area as far as ± 4 km from the borderline. This concept must consider the local physical limitations, including topography as well as the existence of the river and mountains. The border control is provided by the outposts along the inspection belt that keeps track of the country’s resource movement.
Another concept is based on a cluster development that allows the road network to connect with integrated industrial activities (Susantono 2009). In another sense, the concept opens interaction between the activities for production, distribution, and processing of local products. Thus, the of road networks are designed to connect with the adjacent centres in other counties or international transportation hubs as well as to create the internal connectivity of the border region. As illustrated in Figure 2, to serve the industry and trade oriented international market activities, internal and external connectivity are created to support the regional development of an area.

Figure 1. An Illustrative Security and Defence Approach on Road Network Development (Source: Soesetyo, 2009)

Figure 2. An Illustrative Local Economic Development (LED) Approach on Road Network Development (Source: Sambas 2011a)

As oriented in the RTRWN, the concept of growth poles positions the settlement centres in the border region as parts of in the urban system. The road networks in the region are expanded by following the created fabric of space as a National Strategic Area linked with the other centres. This concept shows that the road network development in the border region is not only related to defence and security, but also economic function. The motivations of implementing the concept can be various. One of them is to narrow disparities between the regions in the two countries.

4. ROAD NETWORKS SUPPORTS TO THE BORDER REGIONAL DEVELOPMENT IN SAMBAS

There are two sub-districts constituting the border region of Sambas consisting an area of 2,553.42 km², which are Paloh and Sajingan Besar. The Paloh sub-district manages five villages, meanwhile Sajingan Besar has eight villages under its administration. There are five villages that are close to the borderline. This border region covers about 20% of the total area of the Sambas district, but having only 6 people / km² in its population density (Sambas, 2011a). Based on the District Spatial Plan, the economic development in the area is directed toward small scale industry, tourism, and trade activities oriented to international markets (Sambas 2011a). Those sectors are expected to accelerate the development in the region and to improve the current economic condition.

Regarding the travel behaviour in the region, the two communities in both countries (Indonesia and Malaysia) have a different movement orientation. Sarawak’s border crossers are more based on economic interests or trading, while Indonesian border crossers are motivated by social needs, such as leisure and entertainment. The pattern of cross-border movement between countries is also influenced by social relationships. The majority of community members has relatives and families who are the same (the Malay ethnic). In the general, population in the border region is scattered in many locations. Meanwhile, the outposts connecting the region with Sarawak in Malaysia are located in Temajok and Aruk. Population movements passing the outposts of Aruk to Sarawak for economic purposes are limited to the villagers
coming from Kaliau, Senatab and Sebunga. Most official emigrations to find jobs in Sarawak generally enter through Entikong.

Figure 3. The Study Area is Indicated by Dark Blue Colour (Source: Institute of Road Engineering, 2011)

The Table 1 shows the people’s movement characteristics in the border region of Paloh and Sajingan Besar. There is a wide range of variation among villages regarding their population density, accessibility, mobility, as well as distances to public facilities. Accessibility is indicated by length of road network compared to the village area (km/ km²), while mobility is measured through the number of people living in an area compared to the length of road network. Santosa and Joewono (2005) used indicators, such as road availability (km / km²) and road serviceability (km/ people) to evaluate road network performance in Indonesia. The indicators used in this paper followed the minimum services standard of road networks published by Ministry of Public Works of Indonesia (Minister of Public Works Regulation 2010).

Table 1. The Accessibility and Mobility Characteristics of the Border Region of Sambas

<table>
<thead>
<tr>
<th>Variables</th>
<th>Minimum</th>
<th>Maximum</th>
<th>Mean</th>
<th>Standard Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Population density (people/ km²)</td>
<td>3</td>
<td>90</td>
<td>21</td>
<td>22</td>
</tr>
<tr>
<td>Internal Accessibility (km/ km²)</td>
<td>0.01</td>
<td>0.23</td>
<td>0.08</td>
<td>0.07</td>
</tr>
<tr>
<td>Internal Mobility (people/ km)</td>
<td>1</td>
<td>21</td>
<td>6</td>
<td>6</td>
</tr>
</tbody>
</table>
The Figure 4 and 5 indicate that villages that close to the borderline have good internal accessibility and mobility (showed by the asterisk (*) sign). This contradicts with many public beliefs that the border region has an equal condition and the area that is close to border line usually has lack of road network supports compared to other areas. Since 2009, some efforts have been conducted by the central government to provide the border region of Sambas with adequate roads and other infrastructures. The stages of the development, according to the National Spatial Plan, put this border region as a priority of development until 2012 (Government of Indonesia, 2008). Unfortunately, the road development only focused on the border villages and neglected the connectivity with other villages as an integrated system.

![Internal Accessiblity (km/ km2)](image)

**Figure 4. Internal Accessibility of Each Village in the Border Region of Sambas** (Source: Sambas 2011b)

![Internal Mobility (m / 1,000 persons)](image)

**Figure 5. Internal Mobility of Each Village in the Border Region** (Source: Sambas 2011b)
To measure the differences among villages regarding their characteristics of accessibility and mobility, the Multidimensional Scaling (MDS) technique was used. MDS is also known as perceptual mapping or spatial mapping which is a procedure that can help researchers to determine the relative actual image of a set of objects. The MDS is able to transform stimulus parameters derived from the value of an object through resemblance to the distances represented in multidimensional space.

In this paper, the MDS produced a map that depicts the relative distances of each village using a Euclidean model based on stimuli or variables mentioned in Table 1. Euclidean distance is not always equal to the physical distance, although there may be a comparison of the same distance between them. Distances indicated are similarities between each village according to various parameters submitted accessibility. Greater distances show a large difference, and vice versa. The dimensions produced in the process were evaluated based on the ideal point in each stimulus. By reducing the dimension into two units which is indicated by the change in the Stress (S) value is less than 0.001 (the S value is 0.194), this process resulted that the dimension 1 (the vertical axis) indicates the internal accessibility and mobility, while the dimension 2 (the horizontal axis) is distances to public facilities (education and health). As described in the Figure 6, villages which cluster close to borderline show better access served by the current road networks. In some cases, people’s closeness to public facilities still needs to be improved as shown by Sebunga and Temajuk village.

![Derived Stimulus Configuration](image)

**Figure 6. Euclidean Distance Model Produced by the MDS Analysis**

### 5. KEY STRATEGIES OF ROAD NETWORK DEVELOPMENT IN THE BORDER REGION OF SAMBAS - SARAWAK

The study confirmed some studies that have been done before. BKPMN (2011) indicated that the condition of the border region usually has low accessibility in term of road infrastructure provision with low level access to education and public health. The low accessibility was shown by the availability of road network and the road segments that are in stable condition, therefore the road network system formed the settlements are impartial (Puga, 2008). As shown in Table 3, only half of the villages in the border regions have road segments which are in stable condition more than 75%.

Note:
- VAR 1: Sebunga*
- VAR 2: Kaliau*
- VAR 3: Sanatab
- VAR 4: Santaban
- VAR 5: Sei Bening*
- VAR 6: Kalimantan
- VAR 7: Matang Danau
- VAR 8: Tanah Hitam
- VAR 9: Malek
- VAR 10: Nibung
- VAR 11: Sebulus*
- VAR 12: Temajuk*
- VAR 13: Mentibar
Table 3. Percentage of Road Segments in Good Condition

<table>
<thead>
<tr>
<th>Percentage (%)</th>
<th>No. of Villages</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt; 75%</td>
<td>6</td>
</tr>
<tr>
<td>51-75%</td>
<td>2</td>
</tr>
<tr>
<td>26-50%</td>
<td>2</td>
</tr>
<tr>
<td>≤25%</td>
<td>3</td>
</tr>
</tbody>
</table>

No. of Villages: Kaliau, Santaban, Matang Danau, Tanah Hitam, Temajuk, and Mentibar, Sebunga, Kalimantan, Nibung, Sei Bening, Sanatab, Malek, Sebubus

Source: Sambas 2011b

In 2003, there was a policy which began to notice the development of the border region and put the policy under the term of "frontier area". This policy gives priority to the border region in infrastructure development in contrast to many countries which tend to invest less than the need of their regional development. Puga (2008) elaborated that the common policy arose from a perspective that the growth commonly comes from "spill-over effect" due to the growth of economic activity where locates outside the country’s administration. Thus, it is not seen as an advantage. In another perspective, road networks are considered important to boost the economic activity if supported as well by their road quality. Improvement in road network will enhance people’s well being, distributing economic opportunity and increasing market efficiency (Agrell and Pouyet, 2005).

As Indonesia changed its perspective regarding the border regional development toward economical basis with the neighbour countries, several efforts need to be taken. Firstly, the border region should be developed through infrastructure investment, including road networks that are not only constructed in the areas that locates in the borderline, but also linked them with the activity centres in other villages. The current programmes in infrastructure development of the border region of Sambas heavily focused on the areas at the borderline. This allows extraction of natural resources without enough control by the Sarawak state. The improvement in connectivity is needed to strengthen the concept of local economic development (LED) that was proposed and to form settlement systems as indicated in the RTRWN.

Secondly, the road networks of the region must accommodate the need of local resident to access public facilities. As the settlement spread out in a wide area, investment in road network constructions must be done as a priority. Some areas should be provided with adequate connection to activity centres and public facilities. This can lighten the burden of the local in term cost of transport and time. This strategy views the people’s prosperity in balance with the place prosperity.

As the trade activity of agricultural products from Paloh and Sanjingan Besar to Sarawak intensified, the outpost should be opened to goods movement, following by efforts to improve the CIQS (Customs, Immigration, Quarantine, Security) services (Sihombing 2008). This means that the integration of road networks of the two countries is needed. Therefore, the same standard of road networks as provided in the Annex B of Memorandum of Understanding (MoU) concerning Asean Highway. The way forward to achieve this standard will need time.

6. CONCLUDING REMARKS

The border region of Paloh and Sajingan Besar has followed the path of development as directed in the national policies. In recent years, the central government put some efforts in developing the regions through infrastructure development, including road networks. Unfortunately, the networks are fragmented only focusing on the areas at the borderline while neglecting the connectivity with activity centres in other areas in the region. Some areas have little access to public facilities and roads are in bad condition. If this trend happens for a long time, it is believed that the regional development has more orientation that bring benefits to Sarawak. It also causes the uncontrolled extraction of natural resources, such as raw materials and agricultural products, that has no benefits to the local and the Indonesia’s side.

To integrate the road networks in the region with the Asean Highway, some standards should be met. Considering the recent road quality, the challenges will the topography and soil characteristics of the region. Some areas are dedicated as protected forests that hamper the massive construction of road infrastructure that can disturb the environmental sustainability.
References


Minister of Public Works Regulation No. 14 Year 2010 concerning the Minimum Service Standard for Public Works and Spatial Arrangement Sector


Sambas. 2011. Materi Teknis Rencana Tata Ruang Wilayah Kabupaten Sambas


**PAPER TITLE**
(90 Characters Max)
Development of Warm-Mix Asphalt Technology Applied for Various Types of Asphalt Pavement in Korea

<table>
<thead>
<tr>
<th>TRACK</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>AUTHOR (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yong-Joo KIM</td>
<td>Senior Researcher</td>
<td>Korea Institute of Civil Engineering and Building Technology (KICT)</td>
<td>South Korea</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CO-AUTHOR(S) (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jin-Wook LEE</td>
<td>Research Specialist</td>
<td>Korea Institute of Civil Engineering and Building Technology (KICT)</td>
<td>South Korea</td>
</tr>
<tr>
<td>Kang-Hun LEE</td>
<td>Research Specialist</td>
<td>Korea Institute of Civil Engineering and Building Technology (KICT)</td>
<td>South Korea</td>
</tr>
<tr>
<td>Sung-Do Hwang</td>
<td>Research Fellow</td>
<td>Korea Institute of Civil Engineering and Building Technology (KICT)</td>
<td>South Korea</td>
</tr>
</tbody>
</table>

| E-MAIL (for correspondence) | yongjook@kict.re.kr |

**KEYWORDS:**
warm-mix asphalt (WMA), stone mastic asphalt (SMA) pavement, recycling asphalt pavement and porous asphalt pavement

**ABSTRACT:**
Warm-mix asphalt (WMA) technology was developed to allow asphalt mixtures to be produced and compacted at temperature 30°C lower than conventional hot-mix asphalt (HMA) in Korea. Since a year of 2010, a lots of WMA technologies have been widely commercialized to reduce the mixing and compacting temperatures and provide better compaction on paving construction and ability to haul paving mix for longer distances. These WMA technologies are commonly used in dense-graded asphalt pavement. However, currently, various types of asphalt pavement are used in the asphalt paving industry such as stone mastic asphalt (SMA) pavement, recycled asphalt pavement and porous asphalt pavement. Therefore, the specialized WMA technology for different types of asphalt pavement is needed to develop for easy field applications without any considerations of production and construction processes in asphalt paving industry. Recently, specialized four WMA technologies are developed to use for dense-graded asphalt pavement, SMA pavement, recycling asphalt pavement, and porous asphalt pavement, respectively. This paper presents introduction of each of specialized four WMA technologies with their characteristics and summary of the laboratory evaluation and field implementations, which are conducted for the past five years. On the basis of laboratory and field evaluations of specialized WMA technologies for dense-graded asphalt pavement, SMA pavement, recycled asphalt pavement, and porous asphalt pavement, it is concluded that specialized WMA technologies are effective in producing and compacting four different asphalt mixtures at temperature of 30°C lower than standard hot-mix asphalt (HMA) mixtures. Overall, WMA mixtures show a similar or better performance than standard HMA mixtures in terms of rutting resistance and moisture susceptibility.
Development of Warm-Mix Asphalt Technologies Applied for Various Types of Asphalt Pavement in Korea

Dr. Yong-Joo Kim¹, Mr. Jin-Wook Lee², Mr. Kang-Hun Lee³, Dr. Sung-Do Hwang⁴

¹,²,³,⁴Korea Institute of Civil Engineering and Building Technology, Goyang-Si, Gyeonggi-Do, South Korea
Email for correspondence: yongjook@kict.re.kr

1 INTRODUCTION

Many countries are interested in reducing the emission of green house gasses, which was mainly a result of the Kyoto agreement. In 1997, warm-mix asphalt (WMA) technology was identified as one of means to lower emissions from asphalt industry. According to the reduction of producing and compacting temperatures in asphalt mixtures using WMA technology, the following advantages are obtained: 1) reduction of fuel consumption, 2) less carbon dioxide emission, 3) longer paving season, 4) longer hauling distance, 5) early opening to traffic and 7) a better working environment.

In Korea, annually, about 260-million liter bunker C oil is consumed to produce 30 million tons of hot-mix asphalt (HMA) mixtures. As a result, about 0.8-million CO₂ is produced during the hot-mix asphalt (HMA) production. The asphalt industry considers that warm-mix asphalt (WMA) pavement would be replaced for HMA pavement to offer the potential to lower energy demand during production and reduce emissions at the asphalt plant. In 2010, an innovative WMA technology was developed by Korea Institute of Civil Engineering and Building Technology (KICT). It is named low energy and low carbon-dioxide asphalt pavement (LEADCAP), which is a wax-based composition including crystal controller and adhesion promoter (Kim et al., 2011a & 2011b). Several researchers evaluated LEADCAP WMA technology through the laboratory test and field implementations. Joel et al. (2011) found that the LEADCAP additive could not affect the lower service temperature properties of the binder and, consequently, of the bituminous mixture, while improving the properties of the binder at higher service temperatures. Kim et al. (2011a,) and Lee et al. (2011) found that the LEADCAP additive help improving rutting resistance of WMA mixtures. Yang et al. (2012) conducted mechanical tests, which are including the dynamic modulus test, the direct tension fatigue test, and the triaxial repeated loading permanent deformation (TRLPD) test, in order to evaluate moisture sensitivity, rutting resistance and fatigue life of LEADCAP WMA mixtures. The LEADCAP WMA mixture exhibited better resistance to moisture and rutting than a conventional HMA mixture.

Recently, based on the development of LEADCAP WMA technology, different types of WMA technology, such as SMACAP, RAPCAP, and HiPERCAP WMA additives were developed to use for stone mastic asphalt (SMA) pavement, recycled asphalt pavement and porous asphalt pavement. This paper presents introduction of each of specialized four WMA technologies with their characteristics and summary of the laboratory evaluation and field implementations, which are conducted for the past five years.

2 DESCRIPTION OF SPECIALIZED WMA TECHNOLOGIES

As shown in Figure 1, first, the LEADCAP WMA technology was developed for dense-graded asphalt pavement and then SMACAP, RAPCAP, and HiPERCAP WMA technologies were developed to use in SMA pavement, recycled asphalt pavement and porous asphalt pavement based on the use of concept of LEADCAP WMA technology.

The LEADCAP technology is an organic type of WMA additive, which has a wax-based composition including a crystal controller and an adhesion promoter. The crystal controller adjusts the wax crystallization at the low temperature, preventing the asphalt to show a brittle behavior and the adhesion promoter acts as an effective bonding agent between aggregates and asphalt binder. As a result, it helps improving crack resistance at low temperature and enhancing the moisture susceptibility of WMA mixtures. The LEADCAP additive is melted at about 110°C and can be added to the asphalt mixture (plant-mixed) or to the asphalt binder (pre-mixed) allowing for the production and placement of the mixture at about 30°C below temperatures of a conventional HMA mixture. This additive is typically added at the rate of 1.5% by weight of asphalt binder.
The SMACAP technology is a fiber-combined LEADCAP WMA additive with small amount of asphalt binder for SMA (or PSAM) pavements that require prevention of the binder draindown. The SMACAP additive can be added to the asphalt mixture (plant-mixed) or to the asphalt binder (pre-mixed) allowing for the production and placement of the mixture at about 30°C below temperatures of a conventional SMA (or PSMA) mixture. The SMACAP additive is typically added at the rate of 0.4% by weight of asphalt mixture.

The RAPCAP technology is a liquid-type LEADCAP WMA additive with rejuvenator for recycled asphalt pavements more than 50% use of reclaimed asphalt pavement (RAP) materials. The RAPCAP additive can be added to the asphalt binder (pre-mixed) allowing for the production and placement of the mixture at about 30°C below temperatures of a conventional recycled asphalt mixture. The rate of the RAPCAP additive would be determined by oxidation level and residual asphalt content of RAP materials.

The HiPERCAP technology is SBS-modified LEADCAP WMA additive with dispersant for high viscosity WMA (PG 82-22), i.e., porous asphalt pavement. Dispersant helps that SBS modifier is well distributed with asphalt and aggregate. The HiPERCAP additive can be added to the asphalt mixture (plant-mixed) or to the asphalt binder (pre-mixed) allowing for the production and placement of the mixture at about 30°C below temperatures of a conventional porous asphalt mixture. The HiPERCAP additive is typically added at the rate of 12.0% by weight of asphalt binder.

3 LABORATORY EVALUATION

As shown in Figure 2, wheel tracking test (and mobile moving load simulator (MMLS3) test) and AASHTO T 283 test were conducted to evaluate rutting resistance and moisture susceptibility of WMA mixtures using LEADCAP, SMACAP, RAPCAP and HiPERCAP additives.

For wheel tracking test, two slab samples (300 x 300 x 50 mm) were prepared at 4.0% air void using a linear kneading compactor for each mixture. To measure the deformation over cycle, a rubber wheel was repeatedly moved back and forth on the slab sample at the speed of 42 pass/min with loading level of 690N 40°C. The deformations at 45 minutes and 60 minutes were measured and the dynamic stability was calculated.

For MMLS3 test, two specimens (150 diameter x 5cm height) were prepared at 4.0% air void using a Gyratory compactor for each mixture and individual specimens are clamped properly in test bed. Prior to wheel loading, an initial surface profile is measured from each specimen as a reference point to measure subsequent permanent deformation. The specimens were submerged in heated water at 40°C and the MMLS3 test was conducted at moisture condition.

For AASHTO T 283 test, six specimens (three for dry condition and three for wet condition) for WMA and HMA mixtures were prepared. For dry conditioning, three compacted specimens in a sealed pack were placed in the water bath at 25°C for 2 hours, and for wet conditioning, the other three specimens were placed in the water bath at 60°C for 24 hours followed by conditioning in the water bath at 25°C for 2 hours before the test (AASHTO 2007).
The LEADCAP WMA mixtures along with the control HMA mixtures were evaluated with respect to their rutting resistance and moisture susceptibility (Kim et al., 2011a). As can be seen from Figure 3, the average dynamic stability value of LEADACP WMA mixtures was 3,400 cycle/mm whereas that of the control HMA mixtures was 500 cycle/mm. The average TSR value of LEADCAP WMA specimens was 83% whereas that of the control HMA specimens was 73%. These results indicate that the LEADCAP WMA mixtures would be more resistance to rutting and moisture damage than the control HMA mixtures.

![Figure 3. Laboratory test results of LEADCAP WMA mixtures along with the control HMA mixtures (wheel tracking test (a), AASHTO T 283 test (b))](image)

Figure 4 shows the MMLS-3 test and AASHTO T 283 test results of SMACAP mixtures along with the control hot-stone mastic asphalt (HSMA) mixtures. The average permanent deformation value of SMACAP WMA mixtures was 1.3% whereas that of the control HSMA mixtures was 1.8%. The average TSR value of SMACAP WMA specimens was 82% whereas that of the HSMA specimens was 92%, both mixtures above the specification of 80%.
Figure 4. Laboratory test results of SMACAP WMA mixtures along with the control HSMA mixtures (MMLS3 test (a), AASHTO T 283 test (b))

Figure 5 shows the wheel tracking test and AASHTO T 283 test results of RAPCAP mixtures along with the control hot-recycling asphalt (HRAP) mixtures. As can be seen from Figure 5, the average dynamic stability value of RAPCAP WMA mixtures was 3,150 cycle/mm whereas that of the control HRAP mixtures was 2,333 cycle/mm. The average TSR value of RAPCAP WMA specimens was 85.5% whereas that of the control HRAP specimens was 71.9%. These results indicate that RAPCAP WMA mixtures would be more resistance to rutting and moisture damage than the control HRAP mixtures.

Figure 5. Laboratory test results of RAPCAP mixtures along with the control HRAP mixtures (wheel tracking test (a), AASHTO T 283 test (b))

A Schellenberg drain-down test was conducted to determine whether the amount of drain-down measured for porous asphalt mixture using HiPERCAP WMA additive along with the control hot-porous asphalt mixture using PMA asphalt (HPMA). 1000g of loose mixture are put into an 800ml beaker of known weight, and the beaker containing the mixture is placed in a forced draft oven at 170°C for one hour. At the end of one hour, the beaker containing the mixture is removed from the oven, and loose mixture is removed from the beaker. The weight of remaining materials attached to the wall of the beaker is determined. The percentage of drain-down is then calculated. As shown in Figure 6(a), the average drain-down value of HiPERCAP WMA mixture was 0.05%, whereas that of the control HPMS mixture was 0.18%.

The Cantabro test was conducted to measure the particle loss of compacted HiPERCAP WMA specimen along with the control HPMA specimen. Four Marshall specimens were prepared to run the Cantabro test for each mixture. The Marshall specimens were soaked in water at 25°C for 20 hours, and each specimen was abraded in a Los Angeles abrasion machine. After 300 revolutions, the loose material was discarded, and the weight of the specimen was measured. As shown in Figure 6(b), the average Cantabro loss value of HiPERCAP WMA specimens was 7.0%, whereas that of the control HPMA specimens was 8.8%.

For the resistance to moisture susceptibility, AASHTO T 283 test was conducted. As shown in Figure 6 (c), the average TSR value of HiPERCAP WMA specimens was 91.3% whereas that of the control HPMA specimens was 73.9%.
4 FIELD EXPERIENCES

To investigate workability and compactability of WMA mixtures using LEADCAP additive in the field, as shown in Figure 7, the first LEADCAP WMA field trial sections was built in Korea in 2008. Since then, several LEADCAP WMA field trial sections were successfully constructed in Korea, Portugal, Italy, Japan, United States, Thailand, Chin, Indonesia, and Mongolia. The first RAPCAP WMA field trial section was successfully built in the United States in 2013 (Kim et al, 2011c & 2013 a, b).

Figure 7. Field experiences using LEADCAP and RAPCAP WMA technologies in the world
During the production of LEADCAP WMA mixture at 125±5°C and the control HMA mixtures at 155°C±5°C in the plant, fuel consumption and various emissions were measured. The decreased production temperature by LEADCAP WMA technology leads to energy savings of 32%, which results in 32% reduction of CO$_2$, 18% reduction of CO, 24% reduction of SO$_2$, and 33% reduction of NO$_x$. It indicates that LEADCAP additive would be effective to reduce energy use and the emissions to produce WMA mixtures in the plant.

To compare compactability between LEADCAP WMA and HMA pavement sections, two samples were cored to measure field density and air voids of both pavement sections, respectively. The densities and air voids of LEADCPA WMA and HMA pavement sections were not significantly different in each field experience. It indicates that LEADCAP additive would be effective in compacting WMA mixtures that are comparable to HMA mixtures.

To access the applicability of LEADCAP WMA additive to various asphalt mixtures at different weather conditions, different types of asphalt plant and different mixing methods, LEADCAP WMA additive has been applied to dense-graded asphalt mixture in Portugal, Italy and Indonesia, dense-graded asphalt mixture with polymer-modified asphalt in Mongolia, dense-graded asphalt mixture with reclaimed asphalt pavement (RAP) materials in United States, SBS polymer-modified porous asphalt mixture in Japan, and polymer-modified stone mastic asphalt (SMA) mixture in Chain. RAPCAP WMA additive was applied to dense-graded asphalt mixture with reclaimed asphalt pavement (RAP) materials in United States. The LEADCAP and RAPCAP WMA mixtures were produced at both the batch asphalt plant and drum asphalt plant using pre-mixed method and plant-mixed method under different weather conditions from summer to late winter.

5 CONCLUSIONS

According to the laboratory and field experiences presented in this paper, the following conclusions are drawn:

- Different types of WMA technology, such as SMACAP, RAPCAP, and HiPERCAP WMA additives were successfully developed to use in stone mastic asphalt (SMA) pavement, recycled asphalt pavement and porous asphalt pavement.

- Overall, rutting resistance and moisture susceptibility of the WMA mixture using LEADCAP, SMACAP, RAPCAP, and HiPERCAP additives are similar (or superior) to those of the standard HMA mixture. It indicates that the WMA additive is effective on reducing the production temperature without compromising the mixture performance.

- Based on the experiences of field applications using LEADCAP WMA additive, it is concluded that LEADCAP WMA additive can be used in different plant types under different weather conditions without any problems such as coating, mixing, paving and compacting.

- Given the limited field trials of LEADCAP and RAPCAP WMA pavements, the LEADCAP and RAPCAP WMA pavements achieved a comparable density and air void as a conventional HMA pavement at a significantly lower temperature. The energy savings and the air quality improvements by WMA mixture using LEADCAP additive were observed but long-term performance and durability of LEADCAP WMA pavement should be researched further.

6 ACKNOWLEDGEMENTS

This research is supported by a grant as a Strategic Research Project (Development of Customized Polymer-Modified WMA Technology for Considering Weather and Traffic Conditions in Developing Countries), funded by the Korea Institute of Civil Engineering and Building Technology.

7 REFERENCES


PERFORMANCE-BASED HOT ASPHALT MIX AND FLEXIBLE PAVEMENT DESIGN – THE EUROPEAN PERSPECTIVE

AUTHOR
Ronald BLAB
Prof., Chair for Road and Airfield Engineering
Institute of Transportation, Vienna University of Technology
Austria

CO-AUTHOR(S)
Bernhard HOFKO
Ass. Prof., Head of Asphalt and Bitumen Laboratory
Institute of Transportation, Vienna University of Technology
Austria

E-MAIL
Ronald.Blab@tuwien.ac.at

KEYWORDS:
hot mix asphalt, performance test, mix design, mechanical pavement design

ABSTRACT:
Prediction and optimization of in-service performance of road pavements during their live time is one of the main objectives of pavement research these days. For flexible pavements the key performance characteristics are fatigue and low-temperature, as well as permanent deformation behavior at elevated temperatures. The problem facing pavement designers is the need to fully characterize the complex thermo-rheological properties of hot mix asphalt (HMA) over a wide temperature range on the one hand, while on the other also providing a realistic simulation of the traffic- and climate-induced stresses to which pavements are exposed over their design lives of 20 to 30 years. Where heavily trafficked roads are concerned, there is therefore an urgent need for more comprehensive test methods combined with better numerical forecast procedures to improve the economics and extend the service lives of flexible pavements under repair and maintenance programs.

This papers therefore focus on performance-based test methods on the basis of existing European standards that address effective mechanical characteristics of bituminous materials and which may be introduced into national requirements within the framework of European HMA specifications. These test methods comprise low temperature tests, i.e. the tensile stress restrained specimen test or the uniaxial tensile strength test, stiffness and fatigue tests, i.e. the four point bending beam test or the uniaxial tension compression test, as well as methods to determine permanent deformation behavior by means of dynamic triaxial tests.

These tests are used for the performance-based mix design and subsequently implemented in numerical pavement models for a reliable prediction of in-service performance, which, in combination with performance-based tests, enables a simulation of load-induced stresses and mechanogenic effects on the road structure and thus improved forecasts of the in-service performance of flexible pavements over their entire service lives.
1 INTRODUCTION

For the optimization of flexible road pavements recent research efforts have been focused both on the setup and implementation of performance-based test methods for hot mix asphalt (HMA) as well as on their implementation in valid performance prediction models. While performance-related or empirical tests count for material characteristics that have been found to correlate with fundamental engineering properties that predict performance (e.g. wheel-tracking properties, Marshall properties), performance-based tests describe fundamental engineering properties predicting performance, and appearing in primary performance prediction relationships.

By January 2007 new harmonized European Standards (EN) for the design and testing of road asphalt materials were introduced in all CEN member countries within the European Union. Generally these EN standards distinguish, on the one hand, between the empirical mix design approach and, on the other hand, the fundamental, performance-based approach, which is comparably new. Although both approaches aim in realizing well-performing, structurally optimized pavements, an important advantage of the performance-based approach is the fact that it is based on the laboratory assessment of physically sound material parameters.

These key performance parameters of HMA include (i) complex material stiffness, (ii) fatigue resistance under repeated load cycles (iii) resistance to cracking at low temperatures and (iv) resistance to rutting due to thermal deformation. These material parameters can be used for specifying the mix properties within an advanced type testing procedure required to meet customized quality standards for materials defined in tender documents as well as for mix design (Blab & Eberhardsteiner 2009).

In the European HMA test standard series EN 12697-xx key performance HMA properties are address by different performance tests as summarized in Table 1.

Table 1. European test standards for performance-based tests

<table>
<thead>
<tr>
<th>asphalt course</th>
<th>stiffness</th>
<th>material fatigue</th>
<th>low temperature performance</th>
<th>permanent deformation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface</td>
<td>x</td>
<td>(x)</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>Binder</td>
<td>x</td>
<td>(x)</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>Base</td>
<td>x</td>
<td>x</td>
<td>(x)</td>
<td>(x)</td>
</tr>
<tr>
<td>test procedure</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2-Point-Bending test with trapezoidal specimen (2PB-TR)</td>
<td>Cyclic indirect tensile test (CIDT)</td>
<td>Temperature Stress Test (TSRST)</td>
<td>Triaxial cyclic compression test (TCCT)</td>
<td></td>
</tr>
<tr>
<td>2-Point-Bending test with prismatic specimen (2PB-PR)</td>
<td>4-Point-Bending test (4PB)</td>
<td>Uniaxial tension stress test (UTST)</td>
<td>Uniaxial cyclic compression test (UCCT)</td>
<td></td>
</tr>
<tr>
<td>3-Point-Bending test (3PB)</td>
<td></td>
<td>Uniaxial Cyclic tension stress test (UCTST)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4-Point-Bending test (4PB)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cyclic indirect tensile test (CIDT)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Direct tension-compression test (DCT)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

EN standards: EN 12697-26  EN 12697-24  EN 12697-46  EN 12697-25

x….performance characteristic mandatory, (x)….additional performance characteristic
To identify the rutting behavior at elevated temperatures cyclic axial load tests with or without confining pressures (TCCT Triaxial Cyclic Compression Test or UCCT uniaxial Cyclic Compression Test) are specified. The low temperature behavior is tested by means of the so-called Tensile Stress Restrainted Specimen Test (TSRST) and a Uniaxial Tensile Strength Test (UTST). For characterizing the stiffness and fatigue of asphalt mixtures different tests are described in the European standards, including bending tests (e.g. two point 2PBBT or four point 4PBBT) and direct and indirect tensile tests, but without favoring a particular type of testing device. Further the European HMA specification EN 13108-1 offers different categories for these performance-based HMA properties, which may be introduced as so called fundamental HMA requirements into the national specifications.

Such performance-based HMA specifications, however, require more complex and expensive mix design and type testing procedures. But in combination with these European performance-based HMA specifications mechanistic models allow a more reliable prediction of in-service performance of HMA pavement structures. The objectives of these advanced pavement design models are to enable the simulation of thermo- and load-induced stresses and mechanogenic effects and thus improved forecasts of the in-service performance of flexible and semi rigid pavements.

Following the key performance-based test methods and their possible implementation in mechanistic pavement design models as well as enhanced mix design procedures are discussed in more detail.

2 LOW TEMPERATURE BEHAVIOR

2.1. Background

Traditional studies on pavement performance and modeling have generally concentrated on classical fatigue cracking that consider failure to initiate at the bottom of the bituminous base course induced by a large number of “small” repeated traffic-loadings (Blab & Eberhardsteiner 2009, Wistuba et al. 2006). But this simple approach does not always fit the reality, because two different mayor types of crack damage occur in flexible pavements: cracks that start at the bottom of the bituminous base course and grew upwards, generally named fatigue cracks, and surface cracks that are initiated on top of the pavement. There are always combined effects of critical thermal and load stresses that lead to distress. Thermal stresses are induced by a series of temperature fluctuations within the pavement structure and play a dominant role in the phenomenon of fatigue cracking, and further by a single event, when temperature drops within a very short time. Thermal induced stresses in combination with traffic loading may exceed the critical tensile strength and lead to surface-initiated top-down cracking along the wheel paths.

In the field of low-temperature and fatigue behavior the research activities have been focused on the development of appropriate test methods, to better understand and to identify the fracture mechanisms by means of laboratory experimentation and to further assess the risk of temperature and fatigue cracking for different bituminous materials, which are exposed to stress and temperature.

2.2. Test Methods

Low-temperature cracking of flexible pavements results from thermal-shrinkage during cooling, inducing tensile stress in the asphalt. In order to simulate the situation in flexible pavement layers the following test methods on asphalt specimens according to the European Standard EN 12697-46 are employed:

(i) Tensile Stress Restrainted Specimen Test (TSRST): while the deformation of the specimen is restrained, the temperature is reduced by a pre-specified cooling rate;

(ii) Uniaxial Tensile Strength Test (UTST): in order to assess the risk of low-temperature cracking, the stress induced by thermal shrinkage is compared with the respective tensile strength;

The target parameters, which are found by TSRST, are the fracture temperature ($T_{\text{crack}}$) and the corresponding fracture stress ($\sigma_{\text{crack}}$). An illustration of the test procedure of the TSRST is given in Figure 1.

![Figure 1. TSRST: experimental setup and illustration of result (Blab & Eberhardsteiner 2009)](image-url)
The UTST is an isothermal process at specified temperatures (e.g. +10, +5, -5, -15 and -25°C). After stress-free cooling of the asphalt to the testing temperature, the UTST is performed by applying a constant strain rate (1 mm/min) until the specimen fractures.

Combining the results of TSRST and UTST the tensile strength reserve (Δβ) is found, a “traditional” target parameter for low-temperature cracking (Figure 2).

Figure 2. Superposition of TSRST and UTST results to derive tensile strength reserve (Blab & Eberhardsteiner 2009)

3 STIFFNESS AND FATIGUE BEHAVIOR

3.1. Background

Stiffness and fatigue testing, where a repeated stress is applied on a test specimen, has been a major topic in pavement engineering since decades. Latest research is known from the Association of European Laboratories RILEM and the US Strategic Highway Research Program (SHRP), where a sophisticated layout of different test methods for asphalt concrete design and testing has been developed and which has started a broad discussion on new ways to further optimize fatigue testing procedures and interpretation of test results. Presently two European Standard EN 12697-24 (fatigue) and EN 12697-26 (stiffness) specifies the methods for characterizing the stiffness and fatigue of asphalt mixtures by different tests, including bending tests and direct and indirect tensile tests, but without favoring a particular type of testing device (Di Benedetto et al. 2001, Hofko et al. 2012). However, a single test method for type testing will be imposed on European level in the next future.

3.2. Test Methods

All different types of EN test methods are used to derive basically two material characteristics: the material’s stiffness, expressed by the variation of the complex asphalt modulus (E*(T)) over time, and the long-term fatigue behavior, expressed by the number of permissible load repetitions (Nperm).

The initial stiffness modulus E*(T) of the unloaded material can be determined on the basis of specimen geometry and load impulse and simultaneous measurement of the resulting strains by strain sensors. The stiffness is calculated from the quotient of the applied maximum stress and the resulting maximum strain, which is time-shifted by the corresponding phase displacement angle (φ) as a result of the viscoelastic material behavior of asphalt (Figure 3).

Traditional fatigue criterion of asphalt concrete is linked to the number of load cycles giving half the initial stiffness. The comparison of modulus and the number of load repetitions is plotted as so-called “Wöhler” curve. The Wöhler curve gives important information for the derivation of fundamental relationships between mix composition and stiffness properties and serves as input for material and pavement structure optimization.

Figure 3. Stiffness modulus (E*) and phase angle (φ) (Blab & Eberhardsteiner 2009)
From the EN test methods following two methods were selected to perform stiffness and fatigue tests on asphalt mixtures:

(i) the four-point bending-beam-test (4PBBT) (Figure 4a) and  
(ii) the direct tension-compression test (DTCT). (Figure 4b).

![4PBBT & DTCT equipment](image)

**Figure 4.** 4PBBT & DTCT equipment used for stiffness and fatigue testing (Blab & Eberhardsteiner 2009)

Figure 5 shows typical results of stiffness measurements on a stone mastic asphalt SMA 11 used for wearing courses that were performed at different temperatures and loading frequencies. Results are the master curve of the complex stiffness modulus $E^*$ at reference temperature of e.g. 15°C (Figure 5a), and the frequency independent representation of the loss modulus $E''$ and the conservation modulus $E'$ in a so-called Cole-Cole diagram (Figure 5b). Consequently, these test results describing the temperature and frequency dependent material response of asphalt can be used to compute thermal and load induce stresses and strains in the asphalt layers by means of a numerical pavement model.

![Master curve and Cole-Cole diagram](image)

**Figure 5.** Stiffness master curve of SMA 11 derived form a 4PBBT

For the prediction of the fatigue damage long term tests under repeated dynamic loading are performed. Such tests can be carried out under stress or strain controlled conditions providing typical fatigue curves as given for example in Figure 6 for hot mix asphalt (HMA) AC 22 at 30 Hz and 20 °C. From such curves the permissible load repetitions ($N_{perm}$) are obtained to describe the theoretical life time within an analytically based pavement design method on the basis of fatigue laws.

In the respective EN standard fatigue tests at three different strain levels at 20°C and 30 Hz have to be carried out. Consequently, the allowable strain $\varepsilon_{a}$ at $10^6$ permissible load repetitions is calculated from the semi-logarithmic regression curve as characteristic fatigue parameter of the tested HMA according to Figure 7. In the given example the parameter for the tested AC 22 used for base course layers is $\varepsilon_{a} = 145 \, \mu m/m$. 

404
4 PERMANENT DEFORMATION BEHAVIOR

4.1. Background

Currently one of the main challenging topics in flexible pavement research is the fundamental description of the performance behavior of bituminous mixtures at elevated temperatures. For a better understanding of the permanent deformation behavior tests that realistically simulate in-situ stress conditions and traffic loads are necessary. Permanent deformation can be related to the material characteristics of HMA at hot temperatures in combination with deviatoric stresses and strains under load application. Therefore pavement surface and binder course are most susceptible to permanent deformation. Dynamic or repeated axial load tests with or without confining pressures (unconfined or confined), where these triaxial stress conditions are simulated are considered as most reliable test methods to characteristics the resistance to permanent deformations of bituminous mixtures (Hofko & Blab 2014).

4.2. Test Methods

The triaxial cyclic compression test TCCT was implemented into the series of harmonized European Standards for testing of HMA to assess the resistance to permanent deformation at high temperatures (rutting). The standard test procedure consists of a cyclic dynamic axial loading \( \sigma_A(t) \) to simulate a tire passing a pavement structure and a radial confining pressure \( \sigma_c \) to consider the confinement of the material within the pavement structure. The axial loading \( \sigma_A(t) \) can either be shaped as a sinusoidal function (Figure 8a) or a block-impulse (Figure 8b).

The standard states that the confining pressure \( \sigma_c \) can either be held constant or oscillate dynamically without providing more specific information. However, The TCCT recommended for performance testing is loaded by a sinusoidal axial at a constant confinement loading, respectively.
Figure 9 shows a triaxial testing cell used for permanent deformation tests on HMA. A servo-hydraulic regulated and programmable machine with two independent servo-channels is necessary one to drive the axial loads and the other one for confining pressure. It is possible to run both static tests, i.e. creep tests, and dynamic tests even with dynamic, oscillating confining pressure.

![Triaxial testing cell](image)

Figure 9. Main elements of a triaxial cell used for permanent deformation tests (Blab & Eberhardsteiner 2009)

The axial strain $\varepsilon_N = \varepsilon_{ax}(n)$ is determined for the complete test and drawn in a load-cycle-strain diagram with linear scale for both axes. The resulting creep curve shows two characteristic phases: a primary non linear and a secondary creep phase with a quasi-constant incline of the creep curve. The creep rate $f_c$ in micrometer per meter per load cycle ($\mu m/m/n$) can now be determined as incline of the linear approximation function that is fitted to the quasi-linear part of the creep curve. Figure 10 gives an example for the creep rate $f_c$ calculated for a HMA type used for surface course layers AC 11 with two different binders, one a straight-run bitumen an the other a modified binder.

![Creep curves](image)

Figure 13. Creep curves for HMA type AC 11 surface with two different of binders

Figure 8. a) sinusoidal shaped axial loading and b) block-impulses as axial loading, both with constant confining pressure (EN 12697-25, 2008)
5 HOT MIX ASPHALT REQUIREMENTS

One the basis of these performance based European testing standards different national HMA specifications have been implemented as so called fundamental requirements. Such requirements are no longer recipe orientated that address only volumetric properties (e.g., the air voids (V_a) in the total mix, the voids in the mineral aggregate (VMA), and the voids filled with asphalt (VFA)) of the HMA mixture in terms of measured aggregate and empirical mixture properties but demand performance parameters based on the new European test methods.

Usually different performance parameters are required in dependence on the HMA layer type (base or binder course, wearing course) and the mix design level that is commonly related to the traffic loading class of the pavement.

An example for the performance requirements determined in the Austrian national HMA standard (FSV, 2013) for HMA used in surface, binder courses and base courses are given in Table 2, Table 3 and Table 4, respectively.

Table 2. Performance based requirements for HMA surface layer types AC 11 surf, AC 16 surf, AC 22 surf
(FSV, 2013)

<table>
<thead>
<tr>
<th>parameter/performance level</th>
<th>R1</th>
<th>R2</th>
<th>R3</th>
<th>R4</th>
<th>R5</th>
</tr>
</thead>
<tbody>
<tr>
<td>fracture temperature (T_{crack}) in °C</td>
<td>T_c ≤ -30</td>
<td>T_c ≤ -25</td>
<td>T_c ≤ -30</td>
<td>T_c ≤ -25</td>
<td>T_c ≤ -20</td>
</tr>
<tr>
<td>fatigue resistance ε_ε (Mikrostrain)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ε_ε-NR (no requirements)</td>
</tr>
<tr>
<td>stiffness S_{mix} in MPa</td>
<td></td>
<td>declare S_{min}-value</td>
<td></td>
<td>declare S_{max}-value</td>
<td></td>
</tr>
<tr>
<td>maximum creep rate f_{cmax} in µm/m/n</td>
<td>f_{cmax} ≤ 0,2</td>
<td></td>
<td>f_{cmax} ≤ 0,4</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3. Performance based requirements for HMA binder layer types AC 16 binder, AC 22 binder, AC 32 binder
(FSV, 2013)

<table>
<thead>
<tr>
<th>parameter/performance level</th>
<th>V1</th>
<th>V2</th>
<th>V3</th>
<th>V4</th>
</tr>
</thead>
<tbody>
<tr>
<td>fracture temperature (T_{crack}) in °C</td>
<td>T_c ≤ -25</td>
<td>T_c ≤ -20</td>
<td>T_c ≤ -25</td>
<td>T_c ≤ -20</td>
</tr>
<tr>
<td>fatigue resistance ε_ε (Mikrostrain)</td>
<td></td>
<td></td>
<td>ε_ε ≥ 130</td>
<td></td>
</tr>
<tr>
<td>stiffness S_{mix} in MPa</td>
<td></td>
<td>declare S_{min}-value</td>
<td></td>
<td>declare S_{max}-value</td>
</tr>
<tr>
<td>maximum creep rate f_{cmax} in µm/m/n</td>
<td>f_{cmax} ≤ 0,2</td>
<td></td>
<td>f_{cmax} ≤ 0,4</td>
<td></td>
</tr>
</tbody>
</table>

Table 4. Performance based requirements for HMA base layer types AC 16 base, AC 22 base, AC 32 base
(FSV, 2013)

<table>
<thead>
<tr>
<th>parameter/performance level</th>
<th>E1</th>
<th>E2</th>
<th>E3</th>
<th>E4</th>
</tr>
</thead>
<tbody>
<tr>
<td>fracture temperature (T_{crack}) in °C</td>
<td></td>
<td>T_c ≤ -20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>fatigue resistance ε_ε (Mikrostrain)</td>
<td>ε_ε ≥ 190</td>
<td>ε_ε ≥ 130</td>
<td>ε_ε ≥ 190</td>
<td>ε_ε ≥ 130</td>
</tr>
<tr>
<td>stiffness S_{mix} in MPa</td>
<td></td>
<td>declare S_{min}-value</td>
<td></td>
<td>declare S_{max}-value</td>
</tr>
<tr>
<td>maximum creep rate f_{cmax} in µm/m/n</td>
<td>f_{cmax} ≤ 0,4</td>
<td></td>
<td>f_{cmax} ≤ 0,6</td>
<td></td>
</tr>
</tbody>
</table>

5 CONCLUSIONS

Road constructions today should last longer and endure high traffic loads under challenging climatic conditions. Moreover, traffic densities, axle loads and tire pressures will continue to increase during the next years and decades. To guarantee a long life cycle of flexible and semi rigid pavement structures the optimization of pavement materials in general and bituminous mixtures in particular is getting more and more important in order to avoid damages and subsequently minimize costs for road construction and maintenance.

Therefore prediction of in-service performance of road pavements during their live time is one of the main challenges of pavement research these days. For flexible pavements the key performance characteristics are fatigue and low-temperature, as well as permanent deformation behavior at elevated temperatures. Enhanced test methods, so called
performance based tests, to address these key characteristics are implemented in the latest European standards. So called fundamental requirements for HMA may be specified by the road authorities. These performance tests are used on the one hand to significantly improve the mix design process of bituminous mixtures. On the other hand they provide material input parameters for numerical models that are employed to more reliably predict in-service performance of specific flexible pavement structures.

In combination with enhanced binder tests the implementation of performance-based HMA specifications are the future way to create an innovative road engineering environment in a common Europe.

REFERENCES

Blab R. and Eberhardsteiner J. (2009).” Methoden der Strukturoptimierung flexibler Straßenbefestigungen (Performance-Based Optimization of Flexible Road Structures)”. Progress Report of the CD Laboratory at the Institute of Road Construction, Vienna University of Technology, Vienna. In German


Austrian Transportation Research Society FSV (2013). “Performance Based Specifications for Hot Mix Asphalt Layers, RVS 08.16.06”. Vienna, Austria
# Improving Rutting Resistance and Moisture Susceptibility of Asphalt Binder and Mixtures Using Newly Developed Polymer-Modified Warm-Mix Asphalt Additive in Indonesia

<table>
<thead>
<tr>
<th>PAPER TITLE (90 Characters Max)</th>
<th>Improving Rutting Resistance and Moisture Susceptibility of Asphalt Binder and Mixtures Using Newly Developed Polymer-Modified Warm-Mix Asphalt Additive in Indonesia</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>TRACK</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>AUTHOR (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kang-Hun LEE</td>
<td>Research Specialist</td>
<td>Korea Institute of Civil Engineering and Building Technology(KICT)</td>
<td>South Korea</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CO-AUTHOR(S) (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young-Joo KIM</td>
<td>Senior Researcher</td>
<td>Korea Institute of Civil Engineering and Building Technology(KICT)</td>
<td>South Korea</td>
</tr>
<tr>
<td>Jin-Wook LEE</td>
<td>Research Specialist</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soo-Ahn KWON</td>
<td>Senior Research Fellow</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| E-MAIL (for correspondence) | | yongjook@kict.re.kr |

## KEYWORDS:
Warm mix asphalt pavement, polymer-modified warm mix asphalt, green road, rutting resistance, moisture susceptibility,

## ABSTRACT:

Indonesian government has established the policies of sustainable development for the green road, green city, green building and sustainable transportation. The policies of green road consist of energy saving, recycling, environmentally-friendly construction technologies, and all other things relating to sustainable roads and highways. Recently, Korea Institute of Civil Engineering and Building (KICT) and Institute of Road Engineering (IRE) conducted international joint research in order to develop polymer-modified warm-mix asphalt (PWMA) technology as a sustainable pavement technology in Indonesia. The PWMA technology would resolve distress problems of asphalt pavement in Indonesia such as permanent deformation caused by heavy traffic load and hot temperature and moisture damage caused by raining. This paper presents laboratory test results of PWMA binders and mixtures compared to conventional polymer-modified asphalt (PMA) binder and mixture used in Indonesia. Based on the limited test results, all test results of the PWMA binder is satisfied with Indonesian material specification. Particularly, the performance grade at the high temperature of the PWMA binder is higher than that of typical Indonesian PMA binder. Permanent deformation and stripping resistance of the PWMA mixture are lesser than those of conventional Indonesian PMA mixture. Overall, it can be concluded that the PWMA binder is stronger than conventional Indonesian PMA binder and the PWMA mixture is less susceptible to rutting and moisture damage than conventional Indonesian PMA mixture.
Improving Rutting Resistance and Moisture Susceptibility of Asphalt Binder and Mixtures Using Newly Developed Polymer-Modified Warm-Mix Asphalt Additive in Indonesia

Mr. Kang-Hun Lee¹, Dr. Yong-Joo Kim², Mr. Jin-Wook Lee³ and Dr. Soo-Ahn Kwon⁴

¹,²,³,⁴Korea Institute of Civil Engineering and Building Technology, Goyang-Si, Gyeonggi-Do, Korea
Email for correspondence: kh83lee@kict.re.kr

1 INTRODUCTION

Due to global warming and phenomenon of abnormal temperature, the concern about environmental problem is rising and many of efforts for reducing carbon dioxide is accelerating. In asphalt pavement area, warm-mix asphalt (WMA) has been introduced as eco-friendly pavement technology. In 2006, the WMA technology developed in Europe (Prowell & Hurley 2007) and is known as an alternative technology to replace hot-mix asphalt (HMA). The WMA technology reduces the mixing and compaction temperatures by 20°C to 40°C lower than HMA. The reduction of the mixing and compacting temperatures provides many benefits such as energy saving and emission reduction (Prowell et al. 2012). Currently, WMA-relating technologies are becoming a great item in green road technologies and constructions in the world.

In this trend, Indonesian government has set the policies of developing green road infrastructure, which is defined as a roadway project that has been designed and constructed to a level of sustainability. In 2013, Korea Institute of Civil Engineering and Building Technology (KICT) and Institute of Road Engineering (IRE) conducted international joint research in order to develop green paving materials, which is applicable to WMA additive specialized for heavy traffic road and long-term rainy season in Indonesia. Recently, the first version of polymer-modified warm-mix asphalt (PWMA) additive is developed to use in heavy traffic road at long-term rainy area in Indonesia. The PWMA additive is combined with SBS polymer and wax-based WMA components. This paper presents the laboratory test results of PWMA binder and mixture against conventional Indonesian PMA binder and mixtures. In order to evaluate the asphalt binder using PWMA additive, the following laboratory tests were performed: softening point test, rotational viscosity (RV) test, dynamic shear rheometer (DSR) test of asphalt binder, performance grade (PG) test, and elastic recovery test. Also, compaction test, dynamic immersion test and wheel tracking test was performed to evaluate the compaction effect, moisture sensitive and rutting resistance of the PWMA mixture. The test results of PWMA binder were mostly satisfied with Indonesian PMA specification. The test results of PWMA mixture show better degree of compaction, moisture sensitive and rutting resistance than those of Indonesian PMA mixture.

2 THE CONCEPT OF PWMA TECHNOLOGY

In 2009, Korea Institute of Civil Engineering and Building Technology developed new warm-mix asphalt technology, which is named LEADCAP. It has three components such as polyethylene-based wax, crystallization controller, and adhesion promoter.

Normally, because of its crystallinity, wax based material can be perfectly melted at over melting point. The melting point of the wax used in LEADCAP is around 110°C. LEADCAP additive is quickly liquidized with asphalt
biner when LEADCAP additive is added in asphalt binder at 130~140°C. As a result, liquidized LEADCAP additive can make the viscosity of the binder lower. Generally, wax based WMA additive makes the low temperature properties of asphalt mixture inferior because crystalline wax material is so stiff and brittle at the temperatures below the crystallization point. So asphalt mixtures with a common wax based additive has a high potential for cracking at low temperature. For this reason, LEADCAP additive contains a crystal controller which adjusts the crystalline degree of wax material to avoid becoming too stiff and brittle. The LEADCAP additive contains an adhesion promoter to improve the adhesion between binder and aggregate. As a result, the LEADCAP additive can resist moisture damage without additional anti-stripping agent.

In this study, as shown in Figure 1, Indonesian asphalt binder (pen60/70) with SBS (Styrene-Butadiene-Styrene Block copolymer) and modified LEADCAP additive (LEADCAP-HI) was tested to develop the PWMA which is reasonable for climate and road conditions in Indonesia. The modified LEADCAP additive doesn’t contain crystallization controller because Indonesia weather does not have winter season. The first version of Indonesian PWMA technology focuses on improvement of high temperature property and moisture susceptibility.

![Figure 1. Concept of PWMA additive developed for Indonesian weather and road conditions](image)

### 3 EXPERIMENTAL TESTS OF PWMA ASPHALT BINDERS

For evaluation of Indonesian PWMA additives, as summarized in Table 1, four combinations of SBS, LEADCAP HI, and sulfur materials were selected to evaluate the properties of Indonesian PWMA binder against conventional Indonesian PMA binder. The sulfur is known as additive for improving the strength of asphalt binder (Bencowitz and Boe 1938; Lee 1975). The natural sulfur produced from Indonesia was added to the PWMA binder.

<table>
<thead>
<tr>
<th>Samples</th>
<th>Weight Percentage of SBS</th>
<th>Weight Percentage of LEADCAP HI</th>
<th>Sulfur phr (Part per hundred resin)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HSPW 1</td>
<td>3.5wt%</td>
<td>1.0wt%</td>
<td>3phr</td>
</tr>
<tr>
<td>HSPW 2</td>
<td></td>
<td>1.0wt%</td>
<td>4phr</td>
</tr>
<tr>
<td>HSPW 3</td>
<td></td>
<td>1.5wt%</td>
<td>4phr</td>
</tr>
<tr>
<td>LSPW</td>
<td>3.0wt%</td>
<td>1.5wt%</td>
<td>3phr</td>
</tr>
</tbody>
</table>

Indonesian polymer modified asphalt (Min Pen 40)
For the evaluation of the Indonesian PWMA additives against typical Indonesian PMA, softening point test (ASTM D36), rotational viscosity test (ASTM D4402), DSR test of asphalt binder (ASTM D7175) and performance grade asphalt binder specification without low temperature grade (AASHTO M320) were selected.

![Testing equipment](image)

**Figure 2. Testing equipment for softening point test, dynamic shear rheometer test, and rotational viscometer tests**

Table 2 summarizes the binder test results of Indonesian PWMA additives against typical Indonesian straight asphalt and PMA binder. As can be seen from Table 2, the $G*/\sin\phi$ of the Indonesian PMA satisfied PG 70 as 1.5 at 70°C (Spec: $\geq1.0$ kPa). Thus, the PG of Indonesian modified asphalt at high temperature was determined as 70°C. In addition, $G*/\sin\phi$ of the Indonesia straight asphalt binder satisfied PG 64 as 1.7 at 64°C. The PG Indonesian straight asphalt binder at high temperature was determined as 64°C. This result indicates that Indonesian polymer modifier can improve the asphalt binder as much as one performance grade, from PG 64 to PG 70.

For Indonesian PWMA binders produced by four component combinations, softening points of Indonesian PWMA ranged from 62.6°C to 73°C, all PWMA binders above the specification of 60°C. The PG of Indonesian PWMA binders at high temperature was determined as PG 76. It indicates that the PG of Indonesian PWMA binders at high temperature is one grade higher than that of typical Indonesian PMA binder. It can be postulated that Indonesian PWMA additives would be effective to increase the performance grade at high temperature and rutting resistance of PWMA pavement would be enhanced in the field.

Theoretically, WMA technologies should provide low viscosity for mixing, but polymer modification needs high viscosity for its durability. Therefore, to satisfy both the polymer and warm modifications, the viscosity increased by SBS polymer should be controlled for proper workability. Therefore, the developed Indonesian PWMA additives contain oil and surfactant components to provide lubrication effects as well as wax components to lower its melting point. Based on the needs of low viscosity PWMA binder, LSPW was selected for mix design and performance tests.
Table 2. Test Results of Indonesian PWMA against Indonesian Straight and PMA Binders

<table>
<thead>
<tr>
<th>Reference No.</th>
<th>Test</th>
<th>Softening Point(°C)</th>
<th>Viscosity@135°C (cPs)</th>
<th>G*/sinθ (kPa)</th>
<th>Indonesia PMA</th>
<th>Indonesia Straight Asphalt</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>HSPW 1</td>
<td>63</td>
<td>986</td>
<td>76</td>
<td>1.3</td>
<td>6.0%</td>
</tr>
<tr>
<td>2</td>
<td>HSPW 2</td>
<td>66</td>
<td>1104</td>
<td>1.3</td>
<td>2.2</td>
<td>0.0%</td>
</tr>
<tr>
<td>3</td>
<td>HSPW 3</td>
<td>73</td>
<td>1036</td>
<td>1.5</td>
<td>2.2</td>
<td>0.0%</td>
</tr>
<tr>
<td>4</td>
<td>LSPW</td>
<td>62.6</td>
<td>818</td>
<td>2.4</td>
<td>2.2</td>
<td>0.0%</td>
</tr>
<tr>
<td>5</td>
<td>Indonesia PMA</td>
<td>65</td>
<td>914</td>
<td>1.6</td>
<td>0.5</td>
<td>0.0%</td>
</tr>
<tr>
<td>6</td>
<td>Indonesia Straight Asphalt</td>
<td>50.9</td>
<td>387.5</td>
<td>1.7</td>
<td>-</td>
<td>0.0%</td>
</tr>
</tbody>
</table>

4 MIX DESIGN AND EXPERIMENTAL TESTS OF INDONESIAN PWMA MIXTURES

The laboratory mix design was conducted with LSPW (SBS 3.0wt% + LEADCAP HI 1.5wt% + sulfur 3phr) binder. Based on the criterial of Indonesian design gradation, dense gradation of nominal maximum aggregate size of 13.0mm was selected as shown in Figure 3. The optimum asphalt content was decided as 6.0% with 4.0% air void.

![Figure 3. The Gradation of PWMA mixture and the result of mix design](image)

For evaluation of compactability of Indonesian PWMA mixture, gyratory compaction test (ASTM D6925) was conducted. The Indonesian PWMA mixtures were mixed at 30°C lower than that of typical Indonesian PMA mixtures.

The Indonesian PWMA mixtures were produced at 140–145°C while typical Indonesian PMA mixtures were produced at 170–175°C. Both Indonesian PWMA and PMA mixtures were compacted at 115°C, 135°C and 155°C, respectively. Figure 4 shows a plots of air voids for Indonesian PWMA and PMA mixtures at different compaction temperatures. As can be seen from Figure 4, air voids of Indonesian PWMA mixtures is less than those of Indonesian PMA mixtures. It can be postulated that Indonesian PWMA mixtures would be compacted at 130°C to produce 4.0% air void and Indonesian PWMA mixtures would be compacted at 165°C to produce 4.0% air void.
The dynamic immersion test (DIT) was conducted in accordance with European Standard EN 12697-11 to evaluate the stripping resistance of loose mixtures. For the DIT test, first, the aggregate passing the 11.2mm test sieve and retained on the 8mm sieve was prepared. The mixtures mixed with PWMA and PMA binders were split into three part samples each weight 150±2g after the mixtures were placed in a pan at room temperature for 24 h. Next, 150±2 g of aggregates was placed into a closed 250-mL glass vessel with distilled water. The glass vessel was placed in a rotating device at the speed of 60 rpm at 25°C for 24 h. After the dynamic immersion test was completed. Stripping resistance was estimated by visual observation with individual particles of aggregates. As shown in Figure 5, the PWMA mixtures show less stripped aggregates, while aggregates coated with Indonesian PMA and straight asphalt binders are significantly stripped by water. On the basis of the visual observation, it can be postulated that Indonesian PWMA additive helps increase asphalt aggregate adhesion.

Figure 5. Observation of PWMA, PMA, straight binder mixtures after dynamic immersion test

To evaluate rutting potential of PWMA mixture, the wheel tracking test was performed at 60°C. Two slab samples (300 x 300 x 50 mm) were prepared at 4.0 % air void using a linear kneading compactor for each mixture. To
measure the deformation over cycle, a rubber wheel was repeatedly moved back and forth on the slab sample at the speed of 42 pass/min with loading level of 690N. The deformations at 45 minutes and 60 minutes were measured. As shown in Figure 6, the dynamic stability value of Indonesian PWMA mixtures was 2,885 cycle/mm whereas that of the Indonesia PMA mixtures was 2,735.

![Figure 6. Wheel tracking test results of Indonesian PWMA and PMA mixtures](image)

5 CONCLUSIONS

This paper presents the development of polymer modified WMA additive (PWMA) used for heavy traffic road at rainy area in Indonesia. Based on the test results of PWMA binders and mixtures against typical Indonesian PMA binder and mixtures, conclusions and suggestions are re summarized as below.

1. The polymer modified warm-mix asphalt additive, which are included SBS modifier and LEADCAP HI materials is successfully developed customized for road and weather conditions in Indonesia. LSPW (SBS 3.0wt%+LEADCAP HI 1.5wt% + sulfur 3phr) is selected for the first version of Indonesian PWMA additive.
2. The Indonesian PWMA binder is satisfied with the Indonesia PMA specification. Particularly, the PG of Indonesian PWMA binders at high temperature is one grade higher than that of typical Indonesian PMA binder. It is concluded that Indonesian PWMA additives would be effective to increase the performance grade at high temperature.
3. The PWMA mixtures using LSPW additive can be produced at 140~145°C and compacted at 135°C with 4% air void, which is 20~30°C lower temperature than typical Indonesian PMA mixtures.
4. Based on the visual observation of DIT tested mixtures, the PWMA additive using LSPW additive helps increasing the adhesion between asphalt and aggregate. But this result should be evaluated by another performance tests such as AASHTO T 283 test and Hamburg wheel tracking test.
5. More comprehensive performance tests should be conducted in the laboratory. The field trial section should be built in Indonesia and long-term performance and durability of Indonesian PWMA pavement should be researched further.
6 ACKNOWLEDGEMENTS

This research is supported by a grant as a Strategic Research Project (Development of Customized Polymer-Modified WMA Technology for Considering Weather and Traffic Conditions in Developing Countries), funded by the Korea Institute of Civil Engineering and Building Technology.

7 REFERENCES

BS EN 12697-11 (2003), Determination of the affinity between aggregate and bitumen.
THE ROLE OF CONDITION BENCHMARKING IN ASSET MANAGEMENT,  
CASE STUDY FOR PAVEMENT ASSET IN ABU DHABI – UAE  
Alan Roland, Daniel Ludemann  
Department of Transport – Abu Dhabi, Fugro Middle East – Abu Dhabi  

Summary:  
Best practice asset management involves benchmarking of assets to establish a baseline asset condition to measure and adequately ensure that a continuous improvement cycle is maintained. Infrastructure asset benchmarking measures the service quality, asset performance, condition, safety and environmental effectiveness. Overall it is important to determine the establish the impact of the benchmark results on the level of service and future operations and maintenance costs in order to respond to any relevant system improvements.  

Keywords: Asset Benchmarking, Asset Condition, Quality Assurance/ Quality Control, Remaining Asset Life.  

1 ABSTRACT  
The Department of Transport (DoT) – Main Roads (MR) is managing, operating and maintaining a road network for approximately 10,000 lane.km within Abu Dhabi Emirate in United Arab Emirates. The network is growing rapidly as a result of current capital investment and other improvement projects carried out by the DoT where road projects are vested and handed over by a third party.  
As part of asset benchmarking, four typical applications, including pavement construction quality control/quality assurance (QA/QC) review, pavement layer modulus evaluation, estimation of remaining service life, and mechanistic-empirical design procedures are used to assess asset performance with respect to structural capacity.  
This paper presents a case study where all the above tools are used to evaluate the condition of a recently completed road project, evaluate several quality issues, and the recommendations for mitigating risk of future unforeseen maintenance costs over the asset life.  

2 INTRODUCTION AND BACKGROUND  
The demands and reliance upon the Abu Dhabi pavement system for mobility and commerce have grown substantially over the past few decades and this is expected to continue to grow rapidly with Abu Dhabi’s Vision 2030 plans being executed. As part of the ongoing effort on building and maintaining a world class transport system, the Abu Dhabi Department of Transport maintains a strong focus on assuring the level of service for road users is maintained throughout a pavements lifecycle and has implemented a comprehensive approach for ensuring that both the government and road users in Abu Dhabi are receiving what is anticipated in terms of performance for new pavement infrastructure.  
As part of the pavement network starts to reach the end of the first design life cycle, the pavement owner, which primarily consists of the Abu Dhabi DoT or one of three main Municipalities, are facing conflicting pavement management priorities, constrained budgets, an inefficient construction workforce, and increased risk with respect to project delivery. Whilst the capital works programme for road infrastructure in Abu Dhabi continues to be substantial there is not yet a comprehensive model for road pavement construction projects that addresses some or all of the shortcomings in the traditional road construction model being the sharing of risk between the agency and the contractor(s). The pavement construction and maintenance industry are primarily run through the low bidding procurement system, which has the inherent problem of meeting budget, time, and quality requirements. The low bid contracting system has fundamental difficulties in addressing performance issues.  

3 OBJECTIVE  
As Abu Dhabi DoT is gradually moving toward improved quality assurance and performance-based specifications and contractors are gradually taking a more active role in quality control. The Abu Dhabi DoT process for obtaining product quality involves a combination of following contractor qualification processes:  
1. Pre-qualification (based on having resources, capabilities and item’s 2 and 3 with no or limited rating with quality/price trade off);  
2. Performance Bonds with short term Defects Liability Period;  
3. Third Party Certification (usually ISO 9002);  
4. Project Audits.  
However, these measures do not provide the guarantee that the DoT will obtain the required product quality and furthermore empirical evidence on product evaluation does not show any results of a continual improvement process. So although the term “quality” is used frequently to relate to the contractor qualification process, there is
no evidence of any data or model that establishes relationships between product quality and the current contractor qualification rating system. Therefore, it would be more prudent to reduce or remove the sources of bias in the current conventional qualification system in order to assure high performance.

The term “Performance Based Contracting” is increasingly being adopted in the pavement community, which describes the owner’s requirements in terms of performance as opposed to defining the methods of performance of the work. In the case of pavement contracts the aim of performance requirements is often to minimize the whole of life cost, including construction, maintenance as well as the cost to the road user. Some of the typical performance measures that are adopted for being:

1. Ride Quality of the road surface (typically in terms of the International Roughness Index) which translates to Vehicle Operating Costs;
2. Potholes, cracks and surface wheelpath deformation (rutting);
3. Limits on the friction between the tires and the road surface for safety reasons and relates to accident costs.

The DoT’s expectation is that projects are completed on time, to budget and to a minimum level of quality. Setting aside the safety and lane occupancy cost considerations, as they are a broader discussion and not the focus of this paper, the DoT and user’s expectations for the pavements in Abu Dhabi are to have a smooth ride (i.e. no pavement distress - rutting, potholes and cracking) and adequate friction between tyre and pavement for safety.

The contractor’s expectation is to maximize profit whilst maintaining the probability of winning future contracts. In general, for a healthy contracting system to prevail the owners and users are supposed to get the “best value” of their investment and the contractors are supposed to perform their “best practice” by continuously improving their own performance.

However, what can happen in practice is a process that may become very bureaucratic and the delivery system becomes one of confused liability, having the following characteristics:

1. The designer’s design is reviewed and checked by another designer, confusing the liability issue because a second professional may correct the design;
2. The owner representative directs, manages, and controls the contractor, confusing the liability if there is a performance issue;
3. It is not in the best interest of the designer or contractor to minimize the risk of the owner.

The symptoms present the immediate challenge of how to limit the aforementioned risks by developing a best practice solution that will address the DoT’s objectives in terms of quality and performance for new pavement construction whilst working with the current contracting and specification framework.

4 METHODOLOGY

4.1 An Interim Quality Assurance Solution

As there is an apparent broad and comprehensive structural change required for design, procurement and construction QA/QC framework for pavement assets as well as the implementation of performance based pavement projects, the DoT has implemented a process whereby there is an internal hold point for handover of constructed pavement assets internally. Within the DoT the typical process is to have the Capital Works (Design and Construction Management) Departments “hand over” the completed project to the Operations and Maintenance (O&M) Department for management following the completion of the defects liability period. The O&M Department will then proceed to manage any early maintenance or pavement performance issues and in effect inherits the risk in the medium (>400days) to long term which are not considered in the design or construction process. There are also some examples where an outside entity will construct the road infrastructure as part of a larger project (say a port) and hand over the project to the DoT for management by the Operations and Maintenance Department.

The O&M department has implemented a process of conducting design reviews, data review and pavement testing using qualified and experienced pavement consultants to assess the constructed pavement and assess whether the performance expectations will be met. This is conducted at the start of the defects liability period to ensure that there is adequate time to raise and address issues within the contractor’s liability period. The current term applied to the process is an “asset benchmark and handover” audit.

Various tasks are carried out by the DoT to assess the constructed pavement using a range of state of the practice methodologies and tests. The DoT currently focuses primarily on the following key pavement performance indicators:
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Task</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ride Quality</td>
<td>Pavement Ride Quality Testing (International Roughness Index), Pavement Mechanistic Design Review,</td>
</tr>
<tr>
<td>Rutting</td>
<td>Pavement Mechanistic Design Review, Asphalt Mix Design Review, Structural Evaluation of the Constructed Pavement (Modulus, Remaining Life), Laboratory Testing of Materials (Flow Number)</td>
</tr>
<tr>
<td>Skid Resistance</td>
<td>Pavement macrotexture measurements (Mean Profiled Depth) are taking the DoT assess the pavement frictions characteristics in terms of skidding stopping distance. As a friction policy is under development for the DoT, a friction condition parameter is not included in the evaluation process.</td>
</tr>
</tbody>
</table>

In addition to pavement condition a comprehensive road asset inventory with video imagery is completed as part of the asset benchmark and handover process that complies with the DoT road asset management system requirements. This inventories both condition and asset information for future asset management activities and performance monitoring.

Currently, the Abu Dhabi DoT has adopted the use of an Automatic Road Analyser (ARAN) for carrying out fully integrated surveys to report serviceability parameters, surface defects, asset inventory, and right-of-way imagery. The data for the whole network has been integrated into a web-based application that allows the data and imagery for the entire network to be visible to any user.

![](image)

4.2 Engineering Considerations – Design and Materials

The current specifications in Abu Dhabi for both pavement and materials design are prescriptive and are based on empirical and recipe based methods respectively. Almost all pavements in Abu Dhabi are full depth flexible asphalt pavements. While it could be considered a comprehensive structure the lack of quality control of the materials may comprise the final product performance.

4.2.1 Pavement Design Method

For pavement design the current standard practice is to use the AASHTO Pavement Design Manual 1993 [1]. The AASHTO design procedure is an empirical method and there are significant limitations in using this procedure. As the limitations were not understood then the result was a high risk design, where:
1. The material properties of asphalt layers as well as underlying layers are not directly considered in the structural design process;

2. The types of pavements, the environment and the traffic loading used to develop the AASHTO design procedure differ greatly from the pavements, the environment and the traffic loading found in Abu Dhabi; More specifically:
   a. The environment in Abu Dhabi is significantly different from that of “Illinois”, where the procedure was developed;
   b. Traffic loading in Abu Dhabi is significantly heavier and less controlled than the loading used to develop the procedure;
   c. The pavement materials, in particular the asphalt, are different than that used in the development of the design guide. The binder contents used in Abu Dhabi are significantly lower.

Although the pavement design procedure may be relevant for use in the area it was developed, it is a high risk design method for Abu Dhabi as the method employed has not been developed with the Abu Dhabi traffic, materials or environment as a consideration.

It should be noted that more appropriate mechanistic-empirical models and procedures have been developed and have been tested in similar climatic conditions with similar materials such as the Austroads methods [2].

- Design Traffic

Traffic loads over a design period are the primary parameter that pavements are designed for. When developing a pavement design the designer takes into account the axle loads and operating speeds of traffic for the proposed pavement structure. Abu Dhabi design traffic for the DoT network is characterized by mostly heavy construction vehicles that typically overloaded by international standards which are a key consideration for selecting a pavement design methodology.

4.2.2 Environment

There is currently little or no consideration of the seasonal variation in properties or performance of the various pavement materials or their impact on the pavement performance for the pavement design life. Drainage conditions are assessed as part of the design process, however this does not comprehensively address the seasonal variations in moisture conditions and its impact on pavement performance. Some designers will incorporate a capillary break layer in the design to limit the risk of capillary action for areas with a high water table. There is no consideration of the seasonal temperatures and axle load impact on visco-elastic materials (asphalt mixes) in the current design procedure.

4.2.3 Material Design

4.2.3.1 Subgrade

Subgrade materials are typically characterized as part of a Geotechnical investigation for new pavement construction. This consists primarily of simple laboratory classification tests and bearing strength tests (CBR, DCPT) for design purposes. There are no fundamental tests (resilient modulus) for subgrade conditions however pavement performance is not typically affected by pavement performance. Some discrete areas in Abu Dhabi are affected by “Subkha” which are characterized by very low strength loose fine grained soils where high ground water levels in low lying coastal areas have a substantial effect on the bearing capacity of the subgrade. Typical remedial measures that are introduced include reinforcement (geogrid, geotextile), punched rock platform layers or a combination of these.

4.2.3.2 Unbound Granular Materials

Granular base materials are crushed limestone rock with soil classification, durability and bearing strength (CBR) being the primary specification parameters. These materials are typically quarried or borrowed and for the most part do not present a major risk to the pavement performance as demonstrated by the observed or measured pavement distresses across the Abu Dhabi network.

4.2.3.3 Asphalt, Concrete Mixes

As asphalt mixes almost always form the surface layer, this is the area where the broader quality issues are more apparent to the owner and users in Abu Dhabi. Asphalt concrete mixes are developed using the Marshall Method [3] using modified and unmodified asphalt binders and high quality crushed rock. The established Marshall methodology for mix design has proven to be reliable historically. However, more recent experience and data gathering has shown that asphalt layers are where pavements tend to have quality or premature performance related issues. Furthermore the non-consideration of the asphalt mix properties in the overall pavement design process and vice versa has allowed the contractor to design mixes which are stiff, resistant to short term distress and have inadequate durability meaning medium term risk is not allowed for. However, once it comes to quality control there is a tendency to adopt a recipe based approach to quality control whilst ignoring the fundamental volumetric control considerations.

Additionally, the use of mid-grade penetration graded asphalt binders does not adequately address the extreme
climatic conditions of the region or provide enough characterisation of the binder. Typically, PEN 60-70 or PEN 40-50 binders are utilised, given the climatic conditions, performance graded asphalt binders suited for the temperatures experienced would reduce the risk of material related issues.

4.2.4 Contractor Laboratories

The Abu Dhabi government permits contractors to use their own laboratories in conducting materials quality control testing. In principle, this is an adequate approach to construction quality control, which allows the contractor to control his own product risk. However, in the case of Abu Dhabi these laboratories have not typically been subjected to an adequate audit of their systems and have no internationally recognized accreditation or certification. This presents an overall risk to the project execution and pavement performance as typically when pavement failures occur there will be an inability to rely on the data that has been presented as having been produced by a competent independent certified laboratory. More recently there has been a move to implement and emirate wide wideaccreditation system for laboratories, however this process is in the early stages and the application at the project or site laboratory level is currently not clear.

Additionally, the current practice in the management of quality control data is not effective for engineers to monitor material quality. The quality control results needs to be stored in such a way that is accessible and presented in a manner that easily identifies production trends that can be integrated into a quality management system. For example, the use of control charts that include trend analysis that can help the contractor make corrective action prior to a material falling outside of the specification limits.

4.3 An Engineered Approach to Quality Assurance

The engineering approach for undertaking the asset benchmarking and handover has been developed using a process where the standard practice approach of characterizing the actual constructed pavement is carried out to confirm that the design assumptions have been met i.e. will the constructed pavement be able to carry the expected traffic loads for the design life.

1. Collection of axle load data to develop a design loading based on a comprehensive load spectrum, which takes into account the heavy axle loads in Abu Dhabi.
2. Collection of climatic conditions for Abu Dhabi for both rainfall and temperature.
3. Sampling of the constructed asphalt layers and conducting tests to assess both durability and mix performance for the environmental (temperature) and load conditions that are present in Abu Dhabi.
4. Development of a mechanistic pavement analysis model (as opposed to Empirical) determine the expected pavement life taking into account material parameters, traffic loads and seasonal climatic variations in Abu Dhabi [2].
5. Comprehensive structural evaluation of the constructed pavement using a FWD to verify that the design assumptions developed in 4. Above have been achieved for each pavement layer.

In addition, with respect to monitoring of pavement and materials performance, the pavement condition is “benchmarked” at the start of the defects liability period to identify any quality issues. It is then reassessed towards the end of the defects liability period to check for any premature local or widespread premature failures in the post construction, condition and pavement performance after one season using four key pavement condition and performance parameters (roughness, cracking, rutting and surface macrotexture).

5 RESULTS

5.1 Pavement Performance Issues and Successes

Since the Abu Dhabi DoT has implemented the benchmark and handover process a range of pavement quality issues have been addressed prior to acceptance of the completed project. These include cases such as:

1. Poor ride quality and pavement shape for a newly completed pavement – identified as being an asphalt mix design and placement issue as the root cause. The contractor was then able to remediate the DoT’s satisfaction;
2. Premature rutting of the new pavement section of major project – identified as being a material control issue and the contractor was required to remediate the concerned section with an extended defects liability to monitor other areas of potential concern;
3. Pavement structural design issue for a design and build project developed by another government entity – identified as being an error in the design traffic calculations which meant the pavement structure was under designed by a factor of 5, meaning a 5 year design life as opposed to 25 years as intended.
Table 1: Premature rutting identified during the defects liability period where the Contractor was required to carry out remedial works due to a material issue

<table>
<thead>
<tr>
<th>DIRECTION LANE</th>
<th>RIGHT (Abu Dhabi bound)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>May-11</td>
</tr>
<tr>
<td>10m Lane Segment Averages</td>
<td></td>
</tr>
<tr>
<td>IRI (m/km)</td>
<td>1.07</td>
</tr>
<tr>
<td>Max Rut Depth (mm)</td>
<td>1.42</td>
</tr>
<tr>
<td>Mean Profile Depth (mm)</td>
<td>0.46</td>
</tr>
</tbody>
</table>

In each case the contractor was either required to carry out remedial measures or improvements were recommended for the quality control specification to limit the risk of this happening on future projects.

In addition, there have been cases where it has been demonstrated the contractors have delivered an acceptable pavement product. Projects have included:

1. A major port facility with 330 lane km of pavement developed by another government entity;
2. An 81 km 6 lane expressway link developed by the DoT;
3. A 1km bridge and road link project developed by the DoT.

6 CONCLUSIONS AND OVERALL BENEFITS OF BENCHMARKING

The implementation of the quality benchmarking handover process has provided the DoT with more surety in pavement performance, improved pavement product quality resulting in a more satisfied user and owner of the road asset.

The innovative solutions presented have addressed the challenges of how to limit the systemic pavement construction risk through developing a best practice engineering solution that fits with the DoT’s objectives in terms of quality and performance for new pavement construction whilst working with the current contracting and specification framework.

A multi dimensional benchmarking provides significant benefits not only for the operational planning, but also at the strategic and tactical levels, as it will;

- Target management intervention through the use of a comprehensive mechanistic analysis of pavement performance which is used as a reference for calibration of deterioration models;
- Reduce lifecycle cost through the early identification of pavement construction defects which will fall beyond the defects liability period;
- Increase the usefulness of data whereby the establishment of baseline asset characterisation and inventory can be incorporated into the calibration, asset inventory (location and condition information) and broader decision making process for asset investment;
- Facilitate the ability to compare management and facility issues whereby the agency is able to adapt or modify any part of the design, construction and asset management systems to improve the broader life cycle cost improvements

It is necessary to establish a reasonably standardised basis for comparison. It can be defined as a condition assessment for the final product which is based on the fundamentals of an asset management framework with medium to long term risks as opposed to a process orientated design and construction QA/QC systems which only consider short term risk and require a reference point or justification to implement change.

7 ACKNOWLEDGEMENTS

The authors acknowledging the support from their respective organisations and views expressed in this paper may not represent the organization's views or approach.

8 REFERENCES

BIOGRAPHY

Biographical note for Alan Roland
Alan is a Chartered Professional Registered Engineer of New Zealand (MIPENZ), (CPEng), (IntPE) and REAAA. Member of INGENIUM (Association of Local Government Engineers of New Zealand).
Alan holds a bachelor degree in Civil Engineering, and Masters Degree in Transportation Asset Management. He has experience 30 years in transportation asset management at both strategic and operational levels; Infrastructure development, design, operation and maintenance; Development and implementation of road asset management processes and systems. Alan presented a number of asset management papers at the national and international workshops and conferences.

Alan has been working with government agencies, consultants and contractors, and currently is working with the Department of Transport in Abu Dhabi, as the Network Inventory and RAMS Specialist.

Biographical note for Daniel Ludemann
Daniel is the Division Manager for Fugro’s Pavement Services business activities across the Middle East. Daniel holds a bachelor degree in Civil Engineering from the University of Auckland. He has over 15 years experience in providing and managing pavement engineering consultancy services in both New Zealand and the internationally.

Daniel has provided professional pavement engineering services on a broad range of pavement projects from including pavement design, materials design, forensic investigation, materials specification, quality control and quality acceptance programmes, developed a pavement materials laboratory including a complete SuperPave asphalt laboratory, worked on pavement asset management programmes with comprehensive state of the art pavement evaluation and road network condition monitoring systems. Daniel has provide these services on included major highway construction projects, pavement maintenance programmes and for road network asset and inventory management systems.
PAPER TITLE
(90 Characters Max)
Collecting Pavement Big Data by using Smartphone

TRACK

AUTHOR
(Capitalize Family Name)
YAGI, Koichi
CEO
BumpRecorder Co., Ltd.
Japan

CO-AUTHOR(S)
(Capitalize Family Name)

POSITION
ORGANIZATION
COUNTRY

E-MAIL
(for correspondence)
info@bumprecorder.com

KEYWORDS:
Response type, road profile, big data, disaster response, smartphone

ABSTRACT:
Japan has many earthquakes. In 2011, the Great East Japan Earthquake was occurred, that bring huge damages. On these situation quick survey of road condition is important for quick relief operation. For this survey, convenience method was developed to collect road damages that are the smartphone put on the vehicle dashboard and drive normally. This method is useful not for only relief operation under the disaster, but also for pavement management at usual time.
In this paper, at first, measurement principle of the smartphone application BumpRecorder is explained. This application collect sprung acceleration data from accelerometer of smartphone and position data from GPS. After that vehicle spring condition will be estimated, and then unsprung movement that is wheel axle movement will be estimated. So, this application can be measured pavement roughness without influence of vehicle type and driving speed. And next, an analysis result from collected big data of over 300,000 km is explained. From 2011, the data is collected many places and continuously. The big data usage is explained.
Collecting Pavement Big Data by using Smartphone

YAGI, Koichi

BumpRecorder Co., Ltd., Tokyo, Japan
Email for correspondence: info@bumprecorder.com

1 INTRODUCTION

This bump-collecting project was starting from disaster response usage. Japan is a one of the most highest seismicity country. It is still fresh in memory, at March 11 2011, the grate east Japan earthquake was occurred. During like these situation, immediate survey is an important for relief operation. If information of road damage can be collected conveniently and quantitatively, it will be helpful and effective for relief operation.

Figure 1 shows road damages at Niigata Chuetsu-oki earthquake 2007. An up side photograph was taken at foot of a bridge. This road collapse down, and occurred road bump. A bottom side photograph was taken at near the agricultural land. This road occurred heavy waviness. From this earthquake, our bump-collecting project was starting. In this time, the pedometer was used for bump detector. It was installed on a vehicle dashboard vertical surface. When the vehicle is driven over the road bump, the pedometer is counting up. This counting signal is recording with millisecond timestamp by board computer. At same time, location is recording by GPS. Figure 2 shows an actual installation. By using this data, number of bumps is counting for each 1[km]. This result is drawn on Figure 3. In this figure, a seismic intensity distribution map is used for background map. And cross point indicate the epicenter. Circle makers indicate road bump, and this diameter indicates number of bumps. Look at this figure, this trial method is very simple, and result matches with area damages. It means that vehicle vibration is effective information to evaluate road damage. In this trial, commodity parts were used, but this assembly is specialized system. That is not convenient for spread. From 2007, smartphone was appeared. It has accelerometer and GPS. So smartphone application developing was started.

Figure 1. Road damage at Niigata Chuetsu-oki Earthquake 2007.

Figure 2. Measurement equipment installation.
2 MEASUREMENT METHOD OF BUMP SIZE

When using pedometer, only the number of bumps can be counting. When using smartphone, bump size can be measured. Basic principle of measuring bump size is very simple. Smartphone is installed on the vehicle dashboard, and vertical acceleration is collected. And to get vertical movement, double integral of acceleration is calculated. But in an actual situation, double integral has large cumulative error. It cannot calculate simply. Here, before each integral, values will be corrected as follows.

Step 1: Correcting vertical acceleration. A recording cycle number is defined H[Hz]. At recording order number (i), vertical acceleration is defined $Z(i)[m/s^2]$. A recorded vertical acceleration includes gravity. To calculate dynamic acceleration value $dZ(i)[m/s^2]$, it is assumed that a gravity equal to 10[sec] of acceleration. And next, vertical velocity $Vz(i)[m/s]$ is calculated by summation of dynamic acceleration value $dZ(i)$. That is calculated by equation 1 and 2.

$$dZ(i) = Z(i) - \frac{[\omega = 5Hz]}{H}$$

$$Vz(i) = Vz(i-1) + \frac{dZ(i)}{H}$$

Step 2: Correcting vertical velocity. At uphill and downhill, vertical velocity will be occurred, but it is not occurred by road bump. Uphill and downhill speed is assumed equal to 10[sec] of average speed. And bumping velocity $dVz(i)[m/s]$ is calculated by excluding a average speed. And finally, vertical movement $Lz(i)[m]$ is calculated by summation of bumping velocity $dVz(i)$. That is calculated by equation 3 and 4.

$$dVz(i) = Vz(i) - \frac{[\omega = 5Hz]}{2H}$$

$$Lz(i) = Lz(i-1) + \frac{dVz(i)}{H}$$

Here, bump height is defined by differences between proximity maximum $Lz$ and minimum $Lz$. Bump length is defined by driving distance between above proximity extreme points. When bump height is large, it indicates that this bump is heavy. And when bump length is long, it indicates that this bump is not heavy.
3 EMERGENCY SURVEY AT THE GREAT EAST JAPAN EARTHQUAKE 2011

Using above method, smartphone application named BumpRecorder was provide at March 2nd 2011. One week after, the great east Japan earthquake was occurred. One month after earthquake, our developing team was starting emergency survey at Tohoku region where was concerned damages (YAGI 2011). Figure 4 shows a measurement result on Google Maps. The red circle indicates the epicenter. Blue circles indicate bump positions. This circle diameter indicates sum of square of bump height for each 1[km] long. When there is large bump and there are many bumps, this circle is large. At that time, this map was reported to stakeholders.

Look at this figure large blue circles are concentrated at south part of the epicenter. At this earthquake, terrible Tsunami attacked at north coast of the epicenter, and over 20,000 peoples died at this area. So, many of the news on news papers, TVs were reporting about this north coast area. These reports are important things. But on the other hand, it was not reporting that heavy damages were also occurred at south area by earthquake shake. It is afraid that these situations were influenced for relief operation. Our measurement result could report two month after the earthquake. That was too late to make good effect for relief operation.

A lessons learns from this experience is that penetration of an application and a service are very important for the immediate survey and report. To spread this smartphone application, it starts to apply for a pavement management.

4 SPECIALIZED FOR PAVEMENT MANAGEMENT

To apply pavement management, measurement accuracy is important. When BumpRecorder is used, smartphone is installed in the vehicle cabin for example on the dashboard. Vehicle cabin is located at over the vehicle suspension, a recording acceleration is a sprung acceleration. It is easily influenced from vehicle model and driving speed. Other study reported (Islam 2014), to get good accuracy, vehicle suspension parameter should be including in calculation. To improve measurement accuracy, an estimation method of unsprung vertical movement is developed.
5 ESTIMATION METHOD OF UNSPRUNG VERTICAL MOVEMENT

This estimation method is separated two steps. At the first step, vehicle spring condition is estimated. At the second step, unsprung vertical movement is calculated. And it is assumed that it is equal to road profile, then evaluate this result. The detail is as follows.

(1) Estimation method of vehicle spring condition

The vehicle is modeled by one mass spring model that is drawn in Figure 5. This model is defined by the equation of motion in equation 5. In this equation, \( Lz \) is sprung vertical movement, "\( u \)" is unsprung vertical movement, "\( \omega \)" is angular frequency, "\( h \)" is damping ratio. "\( \omega \)" is defined by equation 6. In this equation, "\( f \)" is a resonant frequency.

\[
\ddot{L}z + 2h\omega (\dot{L}z - \dot{u}) + \omega^2 (Lz - u) = 0
\]  \hspace{1cm} (5)

\[
\omega = 2\pi f
\]  \hspace{1cm} (6)

To calculate this equation, resonant frequency and damping ratio are required. To get these parameters, sprung acceleration is analyzed by FFT, and peak frequency is picked up around 1.5[Hz] for resonant frequency. And using half-width method, damping ratio of this frequency is estimated.

(2) Calculation method of unsprung vertical movement

To calculate the equation 5, \( \ddot{L}z \) is using \( dZ(i) \) of equation 1, \( \dot{L}z \) is using \( dVz(i) \) of equation 3, \( Lz \) is using \( Lz(i) \) of equation 4. Unsprung vertical movement \( u(i) \) is defined by difference equation 7. And \( \dot{u}(i) \) is used for \( \ddot{u} \) of equation 5. \( u(i) \) is used for "\( u \)" of equation 5.

\[
u(i) = u(i-1) + \frac{\ddot{u}(i) + \ddot{u}(i-1)}{2N}
\]  \hspace{1cm} (7)

After applying \( u(0)=0 \), step by step unsprung vertical movement \( u(i) \) is calculated.

6 EXPERIMENTAL RESULT FOR VERIFICATION

(1) Experimental conditions

Accuracy of the proposed method was verified on the road of about 250[m] long at Tsukuba-city, Japan. On this experiment, vehicle speed was changed from 20[km/h] to 60[km/h], and the test drive was done two times for each speed. Figure 6 shows vehicle speed for each drive. Horizontal axis indicates latitude, and vertical axis indicates vehicle speed. The start point is low latitude side or left hand side of this figure, and the end point is high latitude side or right hand side of this figure. Look around the start point and the end point, vehicle speed are not stable. So, period of latitude from 36.1092 to 36.1104 is used for this verification. This period has 160[m] long. On this experiment, for test vehicle, TOYOTA PRIUS was used which has 2,700[mm] wheelbase and 1,400[kg] weight. For test smartphone, Samsung Galaxy S2 was used which has 99[Hz] acceleration sampling cycle that is fastest cycle of this smartphone. And GPS recording cycle is 1[Hz].
(2) Experimental results

Figure 7 shows sprung elevation of each position that is calculated by equation 4. These are measured vertical movement data on the vehicle dashboard. Look at period from latitude 36.1095 to 36.1096, sprung elevation profile of each speed are different. And look at period from latitude 36.1098 to 36.1099, it are different too.

Figure 8 shows unsprung elevation of each position that is calculated by equation 5, 6 and 7. These are estimated vertical movement data at the vehicle axle. Look at above two periods, unsprung elevation differences of each speed are smaller than sprung elevation. It means that unsprung elevation estimation is effective to get good reproducibility. And unsprung elevation can reproduce more pointed profile than sprung elevation profile. It means that unsprung elevation estimation provide higher response. When using unsprung elevation estimation, a vehicle can be driven faster speed with higher accuracy. This result said, this method could reduce spending time and cost for pavement measurement.
To compare both dispersion of the measurement sprung elevation and the estimated unsprung elevation, elevation standard deviation is calculated for each position. This value is drawn in Figure 9. It can easily understand unsprung standard deviation is smaller than sprung standard deviation. In this experiment, mean value of sprung standard deviation is 6.2[mm], and mean value of unsprung standard deviation is 4.4[mm] When using estimated unsprung elevation, standard deviation is reduced to 70%.

![Figure 9. Elevation standard deviation.](image)

7 REPORTING SERVICE

As mentioned above, using for pavement management, accuracy is an important. And unsprung elevation estimation is successfully improving accuracy. One more important issue is a reporting service (Forsløf 2012). Our development team was developing web service named "Bump-Recorder Web". Using smartphone, after measurement road bump, recorded data will be uploaded to web server. Then, unsprung elevation will be estimated on the server. Finally, this data will be displayed on the web browser. From data upload to data display, typically it spends 5 or 10 minutes.

Figure 10 shows this web site screen, on the GSI Maps, that is maps of Geospatial Information Authority of Japan, brown lines draw measured roads, and blue triangular draw measured bumps. A size of the triangular indicates bump height. On this web site, right hand side has a control panel that is used for searching data and analyzing data.

![Figure 10. Sample screen of a bump height.](image)

BumpRecorder estimates an unsprung elevation, and it assumed equal to road profile and calculates IRI, which is an International Roughness Index. Figure 11 shows the sample screen of the IRI with the distance-post graph. On this screen, after selecting two positions, the distance-post graph will be displayed. Figure 12 shows the sample screen of IRI data with the time series graph.
On this website, not only bump height and IRI, but also Japanese flatness named "Heitansei" can be displayed. And analyzing result can be displayed by histogram. In addition for the background map, using not only GSI Maps, but also Google Maps, Google Satellite, and Open Street Map can be selected.

Figure 13 shows the IRI data at Tokyo, Japan. For the north to south direction road that locate at the center of the map, the IRI data was recorded many times. Lower right graph shows the IRI data distance-post graph for north to south direction. Data were recorded from March 2011 to July 2014. First half data before November 2012 is drawn by orange line. Second half data after November 2012 is drawn by green line. These two data have same trend. It means that road profile is not changing.

Figure 14 shows opposite direction, south to north direction. At distance from 100 to 600, orange line data are large than green line data. It means that this road was repaired and road profile was improved.

These situations can be found by using collecting pavement big data from smartphone. In Japan, collecting bump data is over 450,000[km] long, it is becoming pavement big data.

8 NEW TYPE NAVIGATION - BUMP NAVI -

This pavement big data bring new type of navigation that is bump navigation. Smartphone application Bump Recorder has [Navi] mode screen. Figure 15 shows sample screen of bump navigation. On this screen, your position is drawn by light blue circle, and road bump and IRI are also displayed. Using this navigation, it is easy to find the road bump before vehicle bumping. Especially, driving an ambulance car or delivering track of fragile articles or precision machines, this navigation is very useful and effective.
9 PAVEMENT BIG DATA REVOLUTIONS

Smartphone type road roughness measurement has cost merit. But it is just a one side. It will bring several revolutions. When a road roughness is measured by specialized vehicle, special technics are required to the measurement operators. Therefore, frequent measurement is difficult not only at the budget side but also at the human resource side. So it is measured only one year once or few years once. That is not enough to pavement maintenance management.

When it is using smartphone, measurement operation is easier than using specialized equipment. So, road roughness measurement will be possible, not only by measurement specialist, but also by local government staffs, company drivers and citizens. Spreading of measurement operators are bringing high frequent measurement that is monthly, weekly, daily or more, instead previous yearly measurement. At same time, measured road will be increased. As a result, both of the time based and the distance based measured data volume will be exploding.

For this data exploding, measurement convenience is very important. Using smartphone is one thing, but it is not enough. If the smartphone installation is required perfect gravity direction, if it is required calibration before measurement, if it is required information input of vehicle model etc., or if it is required driving with same speed at all the time, whenever using smartphone, it is not convenience. In this result, data volume cannot be exploding.

To improve these issues, the proposed smartphone application BumpRecorder only required smartphone is installed on the vehicle tightly and driving more than few km in one measurement, but not required other conditions. This application has an automatic estimation function of installation direction, so it is possible to install at any direction. It has an automatic estimation function of spring constant of vehicle suspension, so it is not required calibration driving before measurement driving, and it can drive normally with usual speed up and speed down. These functions are very important to increase measurement cooperators. As a result, measurement data volume will be increased.

There is one more important thing to increase cooperators. That is cooperators merits. For pavement maintenance managers, road roughness measurement is an important thing. But for other peoples, it is not so important. In other words, they have not motivation for measurement. To improve these things, other useful information should be picked up from same data that are GPS data and acceleration data. It is easy to conceive that GPS data real time sharing is useful for location services. When the bus service company becomes cooperators, bus location service can be made. When the taxi or freight service company becomes cooperators, location services can be made to help dispatch control. When there are many measurement cooperators, traffic information can be made. In addition, at winter, when snowplow has location service, citizens will know which road is finished snow removal. This information is important for safety.
driving. In addition, not only shared GPS data, but also acceleration data in real time, it can estimate which road is become pressed-snow, because pressed-snow road will become rougher road. Like these multi purpose usage is bringing new merits for many peoples and it will increase cooperators and it will increase collected data volume.

Proposed smartphone application BumpRecorder has a real time data sharing function. Bus location service is become a starting service, provide services will plan to spread to snowplow location service, taxi, truck location service, and etc. And finally it will spread to traffic information in future days. Through these multiple useful services, creating pavement big data will become a reality.

Here again, most important thing is a convenience. Using smartphone is just a starting point. Adjustment free, calibration free, multiple services are much important. When it becomes a reality, road roughness measurement and pavement maintenance management will have big revolutions.

REFERENCES


ABSTRACT:
Generally, a failure of asphalt pavement occurred during the rainy season. This indicates that the vulnerability of asphalt pavement is due to the water entered into the pavement structure. To solve the problems, water damaged of asphalt pavement can be minimized by adding anti-stripping agent to increase the adhesiveness of asphalt to aggregate. Therefore the asphalt pavement becomes resist to strip when submerged in water. Based on chemical analysis, Asbuton (natural rock asphalt from Buton Island) is also potential to be used as anti- stripping agent because the Asbuton consists of bitumen with resins content is around three times higher than petroleum asphalt (Hermadi et al , 2012). In addition, Oyekunle (2005) stated that, resin is not just providing ductility, malleability and plasticity, but also increases the asphalt adhesion to aggregate. In this paper, how Asbuton can act as an anti-stripping agent is presented. The method that used is by comparing between the effect of water on properties of un-modified asphalt mixtures, asphalt mixture with liquid anti-stripping agent around 0.3% of bitumen, and asphalt mixture with Asbuton Pellets around 2.5% of mixture. The mixture properties included Marshall properties and Tensile Strength Ratio (TSR). Finally, the results showed the TSR of the un-modified asphalt mixture is 87.3%, the asphalt mixture with liquid anti-stripping is 89.1 % and the asphalt mixture with Asbuton Pellets is 89.5%. Thus, Asbuton Pellets can be used as anti-stripping agent of asphalt mixture.
Utilization of Asbuton As an Anti-Stripping Agent of Asphalt Pavement

Madi Hermadi¹, Kurniadjı²

¹,² Center for Research and Development for Roads and Bridges, Indonesia

Email for correspondence: madi.hermadi@gmail.com

1 Introduction

Global warming has changed weather patterns in Indonesia. Indonesia, that has a tropical climate, normally every half year change from rainy season to dry season or otherwise. The last few years it turned into a rainy season throughout the year. It leads to flooding in some areas, including in Jakarta. These conditions also resulted in the performance of asphalt pavement that often damaged because of stripping. To overcome this, the Directorate General of Highways has issued pavement specifications that require the use of anti-stripping agent in the asphalt mixture.

Anti-stripping agents are used the world-over, to overcome the stripping problem in bituminous pavements because anti-stripping agents improve the bond between asphalt binder and aggregate (BSI, 2001). Molecule of anti-stripping agents consists of hydrophilic and hydrophobic parts. The hydrophilic parts that also called polar parts have an affinity for water and bond tightly with aggregate meanwhile the hydrophobic parts that also called non-polar parts have an affinity for oil and bond tightly with asphalts. Therefore the anti-stripping agents act as a bridge between the ‘water-loving’ polar aggregates and ‘water-hating’ non-polar asphalts (Gore, 2005). Currently, the most commonly used anti-stripping agents are long chain organic amines of relatively high molecular weight such as hydrocarbons fatty diamine, fatty amido-diamine, etc because they are the most suitable anti-stripping agents for bitumen (BSI, 2001).

However, Oyekunle stated that not only alkylation nitrogen based that can act as an anti-stripping materials, but also asphaltenes and resins can act as an anti-stripping materials. It because the asphaltenes and resins are hydrocarbon compounds which are more polar asphalt components than the other components (aromatics and paraffins). Based on the Oyekunle statement, an idea was obtained to examine Asbuton for used as an anti-stripping agent of asphalt, since Asbuton bitumen has asphaltenes and resins contents higher than petroleum asphalt. Based on chromatography test results in Japan, the asbuton bitumen consists of asphaltenes and resins around two and three times more than petroleum bitumen respectively (Hermadi, 2013).

Etymologically, Asbuton derived from bahasa Aspal Batu Buton that means Buton Rock Asphalt. Terminologically, Asbuton is natural rock asphalt from Buton Island in Southeast Sulawesi Province in Indonesia. Asbuton have a very large deposit of around 700 million tonnes with a bitumen content of around 20% and the remaining minerals with the largest component of limestone.

Because Asbuton has high possibility to be used as an anti-stripping agent, this study investigates the effect of the use of Asbuton on striping susceptibility of hot mix asphalt.

2 Literature Review

There are many factors cause stripping or loss of adhesion between the aggregate surface and asphalt cement binder of hot mix asphalt such as the presence of moisture, aggregate properties, asphalt cement binder properties, mixture characteristics, climate, traffic, construction practices, and pavement design considerations (Little and Apps, 2001). In Indonesia, presence of moisture is very significant factor because of tropical climate with high rainfall. Furthermore, climate change as an effect of global warming has led in recent years occurred rainy season throughout the year and led to frequent flooding in many places including in Jakarta.

One of the ways to overcome the stripping damage of asphalt pavement is by adding antistripping agents such as fatty amines, fatty amido-amines, fatty imidazolines, and hydrated lime. Antistripping agents are surfactants with two kinds of groups, one group has binder affinity (lipophilic or non-polar) and the other has water or polar substances affinity (hydrophilic or polar). In asphalt pavement, the antistripping agents place themselves in the bitumen-aggregate interface to increase the adhesion (Castano, 2004).

According to Oyekunle (2006), asphaltenes and resins in asphalt binder also act as antistripping agents. Asphaltenes are the most complex molecules present in asphalts and are regarded as being formed by the condensation of resins. They are black or brown colored, hard, non-plastic, nonmalleable, high molecular weight compounds ranging between 1200 and 200,000. They contain predominantly carbon and hydrogen with sulfur, oxygen, nitrogen and other heteroatoms. Asphaltenes are agglomerations of the most highly polar molecules and they are responsible for the presence of structure in asphalts. They are insoluble in low molecular weight normal paraffins and are classified by the precipitating solvent; different solvents precipitating different amount of asphaltenes. They impart strength, stiffness and colloidal structure in asphalt. They are determined by ASTM D3272 as the n-heptane insolubles. Furthermore, resins are semiliquid and sometimes solid materials of dark red color at room temperature. They are chemically very similar to the asphaltenes; present in resins are carbon, hydrogen, oxygen, sulfur, nitrogen and many other elements including...
metals. The resins consist of mainly polycyclic molecules containing saturated, aromatic and hetero-aromatic rings and heteroatoms in various functional groups. The resins are not as polar as the asphaltenes and their molecular weight ranges from 300 to 2000. These resins provide adhesion, ductility, malleability and plasticity.

Hermadi (2013) explained, Asbulon Lawele and Kabungka consisted of bitumen that rich with asphaltenes and resins. They are around three times of petroleum asphalt as shown in Figure 1. Accordingly, both Asbulon are potential to be used as an antistripping agent.

![Figure 1. Chemical composition of petroleum asphalt, Lawele Asbulon and Kabungka Asbulon bitumens (Hermadi, 2013)](image)

3 MATERIALS AND METHOD

The materials that used in this study are petroleum bitumen penetration grade 60, aggregates (Coarse, medium and fine), Asbulon Pellets and antistripping agent with properties as exhibited in Table 1, Tabel 2, Table 3, Table 4 and Table 5 respectively. Accordingly, the used petroleum bitumen and aggregates are meet the specifications that mean the materials are feasible to be used as road materials. The used Asbulon is Kabungka Asbulon pellets that consist of around 35% bitumen with bitumen penetration around 8 dmm. The antistripping that used is amine-based anti stripping with amine value more than 200.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Test Method</th>
<th>Result</th>
<th>Specification(^1)</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Penetration at 25°C, 100g, 5 sec</td>
<td>SNI 2456 : 2011</td>
<td>65</td>
<td>60 - 70</td>
<td>dmm</td>
</tr>
<tr>
<td>Viscosity at 135°C</td>
<td>SNI 06-6441-2000</td>
<td>375</td>
<td>≥ 300</td>
<td>eSt</td>
</tr>
<tr>
<td>Softening point</td>
<td>SNI 2434 : 2011</td>
<td>48,2</td>
<td>≥ 48</td>
<td>°C</td>
</tr>
<tr>
<td>Ductility at 25°C, 5 cm/min</td>
<td>SNI 2432 : 2011</td>
<td>&gt; 140</td>
<td>≥ 100</td>
<td>Cm</td>
</tr>
<tr>
<td>Flash point (COC)</td>
<td>SNI 2433 : 2011</td>
<td>288</td>
<td>≥ 232</td>
<td>°C</td>
</tr>
<tr>
<td>Solubility in C(_2)HCl(_3)</td>
<td>SNI 06-2438-1991</td>
<td>99,813</td>
<td>≥ 99</td>
<td>%</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>SNI 2441 : 2011</td>
<td>1,034</td>
<td>≥ 1.0</td>
<td>-</td>
</tr>
<tr>
<td>Loss on heating (TFOT)</td>
<td>SNI 06-2440-1991</td>
<td>0.0194</td>
<td>≤ 0,8</td>
<td>%</td>
</tr>
<tr>
<td>Penetration at 25°C, 100g, 5 sec after TFOT</td>
<td>SNI 2456 : 2011</td>
<td>83,1</td>
<td>≥ 54</td>
<td>% original</td>
</tr>
<tr>
<td>Softening point after TFOT</td>
<td>SNI 2434 : 2011</td>
<td>52,1</td>
<td>-</td>
<td>°C</td>
</tr>
<tr>
<td>Ductility at 25°C, 5 cm/min after TFOT</td>
<td>SNI 2432 : 2011</td>
<td>&gt; 140</td>
<td>≥ 100</td>
<td>Cm</td>
</tr>
<tr>
<td>Mixing temperature</td>
<td>ASSHTO-72-1990</td>
<td>153 - 159</td>
<td>-</td>
<td>°C</td>
</tr>
<tr>
<td>Compaction temperature</td>
<td>ASSHTO-72-1990</td>
<td>140 - 146</td>
<td>-</td>
<td>°C</td>
</tr>
</tbody>
</table>

\(^1\)The Directorate General of Highways (2010)
Table 2. Properties of the coarse, medium and fine aggregates

<table>
<thead>
<tr>
<th>Properties</th>
<th>Method</th>
<th>Test Result</th>
<th>Specification</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Coarse</td>
<td>Medium</td>
<td>Fine</td>
</tr>
<tr>
<td>Specific gravity:</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Bulk</td>
<td>SNI 03-1969-2008</td>
<td>2.63</td>
<td>2.58</td>
<td>2.66</td>
</tr>
<tr>
<td>• SSD</td>
<td>&amp;</td>
<td>2.68</td>
<td>2.64</td>
<td>2.70</td>
</tr>
<tr>
<td>• Apparent</td>
<td>SNI 03-1970-2008</td>
<td>2.77</td>
<td>2.74</td>
<td>2.77</td>
</tr>
<tr>
<td>Water absorption</td>
<td>SNI 03 1969-2008</td>
<td>1.9</td>
<td>2.3</td>
<td>1.6</td>
</tr>
<tr>
<td>Coating and stripping</td>
<td>SNI 2439:2011</td>
<td>95+</td>
<td>Min 95</td>
<td></td>
</tr>
<tr>
<td>Sieve analysis:</td>
<td>ASTM C136-12</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• # 3/4&quot; (19,1 mm)</td>
<td>100</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• # 1/2&quot; (12,5 mm)</td>
<td>93</td>
<td>100</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• # 3/8&quot; (9,5 mm)</td>
<td>33</td>
<td>97</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>• # No. 4 (4,76 mm)</td>
<td>1.9</td>
<td>24</td>
<td>99</td>
<td></td>
</tr>
<tr>
<td>• # No. 8 (2,36 mm)</td>
<td>1.7</td>
<td>5.6</td>
<td>74</td>
<td></td>
</tr>
<tr>
<td>• # No. 16 (1,18 mm)</td>
<td>1.5</td>
<td>3.9</td>
<td>45</td>
<td></td>
</tr>
<tr>
<td>• # No. 30 (0,60 mm)</td>
<td>1.4</td>
<td>3.0</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>• # No. 50 (0,30 mm)</td>
<td>1.3</td>
<td>2.3</td>
<td>21</td>
<td></td>
</tr>
<tr>
<td>• # No. 100 (0,149 mm)</td>
<td>1.1</td>
<td>1.6</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td>• # No. 200 (0,075 mm)</td>
<td>0.8</td>
<td>0.9</td>
<td>11</td>
<td></td>
</tr>
</tbody>
</table>

1 The Directorate General of Highways (2010)

Table 3. Properties of Asbuton Pellets

<table>
<thead>
<tr>
<th>Properties</th>
<th>Test Method</th>
<th>Results</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bitumen content</td>
<td>SNI 03-3640-1994</td>
<td>35,4</td>
<td>%</td>
</tr>
<tr>
<td>Water content</td>
<td>SNI-2490-2008</td>
<td>0,1</td>
<td>%</td>
</tr>
<tr>
<td>Sieve analysis of Asbuton mineral:</td>
<td>ASTM C136-12</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• # No. 16 (1,18 mm)</td>
<td>100</td>
<td>% Passing</td>
<td></td>
</tr>
<tr>
<td>• # No. 30 (0,60 mm)</td>
<td>94</td>
<td>% Passing</td>
<td></td>
</tr>
<tr>
<td>• # No. 50 (0,30 mm)</td>
<td>74</td>
<td>% Passing</td>
<td></td>
</tr>
<tr>
<td>• # No. 100 (0,149 mm)</td>
<td>68</td>
<td>% Passing</td>
<td></td>
</tr>
<tr>
<td>• # No. 200 (0,075 mm)</td>
<td>60</td>
<td>% Passing</td>
<td></td>
</tr>
</tbody>
</table>

Table 4. Properties of Asbuton pellets bitumen

<table>
<thead>
<tr>
<th>Properties</th>
<th>Test Method</th>
<th>Test Result</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Penetration at 25°C, 100g, 5 sec</td>
<td>SNI 2456 : 2011</td>
<td>8</td>
<td>dmm</td>
</tr>
<tr>
<td>Softening point</td>
<td>SNI 2434 : 2011</td>
<td>73,2</td>
<td>°C</td>
</tr>
<tr>
<td>Ductility at 25°C, 5 cm/min</td>
<td>SNI 2432 : 2011</td>
<td>0</td>
<td>Cm</td>
</tr>
<tr>
<td>Flash point (COC)</td>
<td>SNI 2433 : 2011</td>
<td>268</td>
<td>°C</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>SNI 2441 : 2011</td>
<td>1,141</td>
<td>-</td>
</tr>
<tr>
<td>Solubility in C₆H₅Cl₂</td>
<td>SNI-06-2438-1991</td>
<td>99,50</td>
<td>%</td>
</tr>
<tr>
<td>Loss on heating (TFOT)</td>
<td>SNI 06-2440-1991</td>
<td>0,544</td>
<td>%</td>
</tr>
<tr>
<td>Penetration at 25°C, 100g, 5 sec after TFOT</td>
<td>SNI 2456 : 2011</td>
<td>62,5</td>
<td>% original</td>
</tr>
<tr>
<td>Softening point after TFOT</td>
<td>SNI 2434 : 2011</td>
<td>75,1</td>
<td>°C</td>
</tr>
<tr>
<td>Ductility at 25°C, 5 cm/min after TFOT</td>
<td>SNI 2432 : 2011</td>
<td>0</td>
<td>Cm</td>
</tr>
</tbody>
</table>
Table 5. Typical properties of the anti-stripping agent

<table>
<thead>
<tr>
<th>Properties</th>
<th>Test Method</th>
<th>Typical value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flash point</td>
<td>SNI 03-6722-2002</td>
<td>&gt; 200</td>
<td>°C</td>
</tr>
<tr>
<td>Pour point</td>
<td>ASTM D97 - 12</td>
<td>&gt;15</td>
<td>°C</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>ASTM D3142-11</td>
<td>&lt; 1.02</td>
<td></td>
</tr>
<tr>
<td>Amin value</td>
<td>ASTM D2074 - 07</td>
<td>&gt; 200</td>
<td>mg KOH/g</td>
</tr>
<tr>
<td>Water content</td>
<td>SNI 2490:2008</td>
<td>&lt; 1</td>
<td>%</td>
</tr>
</tbody>
</table>

4 METHOD

The effect of Asbuton pellets on stripping susceptibility of four types asphaltic mixtures were investigated. The mixtures are:
1) Type 1 is hot mix asphalt mixture with the petroleum bitumen pen 60 as a binder.
2) Type 2 is hot mix asphalt mixture with the petroleum bitumen pen 60 plus 0.3% anti-stripping agent as a binder.
3) Type 3 is hot mix asphalt mixture with the petroleum bitumen pen 60 plus Asbuton pellets (the ratio of petroleum asphalt to Asbuton pellets bitumen is 85:15 or equal with 2.5% of the mixture) as a binder.
4) Type 4 is hot mix asphalt mixture with the petroleum bitumen pen 60 plus 0.3% antistripping agent and plus Asbuton pellets (the ratio of petroleum asphalt to Asbuton pellets bitumen is 85:15 or equal with 2.5% of the mixture) as a binder.

The investigation steps for all the mixtures were:
1) Mix design that used Marshall Method to find out the optimum composition. Mixing and compaction temperatures were 156 °C and 143 °C respectively. The temperatures revere to the petroleum bitumen properties as shown in Table 1. Compaction was done by 2 x 75 blows of Marshall Compactions.
2) Then, some briquettes of each optimum mixture were tested in properties of Volumetric, Marshall Stability, Flow, Retained Stability after immersion in water and Tensile Strength Ratio (TSR).

5 RESULTS AND DISCUSSION

5.1 Mixture gradation

The aggregate composition of type 1 and type 2 mixtures that meets the specification of AC-WC gradation are as follow and the gradation as shown in Figure 1.

- Coarse aggregate : 25,0%
- Medium aggregate : 28,0%
- Fine aggregates : 46,0%
- Cement filler : 1,0%
The aggregate composition of type 3 and type 4 mixtures that meets the specification of AC-WC gradation are as follow and the gradation is as shown in Figure 2.

- Coarse aggregate : 25.0%
- Medium aggregate : 30.0%
- Fine aggregates : 44.4%
- Cement filler : 1.0%
- Asbuton pellets filler : 1.6%

Even though the mixture with 0.3% anti-stripping in the mixtures had produced Marshall Stability of the mixture became significantly lower. This condition is caused that the anti-stripping is liquid and its make the asphalt becomes softer, and it is in accordance with the slightly changes in the flow and the Marshall Quotient which indicates the mixture with 0.3% anti-stripping is more flexible.

Furthermore, Type 1 mixture has lowest retained stability after soaking (84.2 %) and Tensile Strength Ratio (TSR = 87.3 %). It is also lower than minimum requirement of Indonesian Specification of Hot Mix Asphalt that minimum 90% for retained stability and TSR (The Directorate General of Highways, 2010). Therefore, the mixture should be modified that in this case the modifying was by adding anti-stripping agent and Asbuton Pellets. The results, retained stability and TSR of the mixture increased to become 88.4 % and 89.1 % respectively by adding anti-stripping agent 0.3 % of bitumen (Type 2 mixture), to become 87.9 % and 89.5 % respectively by adding Asbuton Pellets 2.5 % of the mixture (Type 3 mixture), and to become 94.0 % and 95.1 % respectively by adding anti-stripping agent 0.3 % of bitumen and Asbuton Pellets 2.5 % of the mixture (Type 4 mixture). The differences of retained stability and TSR value of the mixtures are shown in Figure 4.
Table 6. Mixture Properties at Optimum Bitumen Content

<table>
<thead>
<tr>
<th>Properties</th>
<th>Test Method</th>
<th>Test Results</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Optimum bitumen content</td>
<td>SNI 06-2489-1991</td>
<td>6.1 6.1 6.1 6.1</td>
<td>%</td>
</tr>
<tr>
<td>Effective bitumen content</td>
<td>SNI 06-2489-1991</td>
<td>5.8 5.8 5.8 5.6</td>
<td>%</td>
</tr>
<tr>
<td>Density</td>
<td>SNI 03-6756-2002</td>
<td>2.33 2.33 2.33 2.34</td>
<td>ton/m³</td>
</tr>
<tr>
<td>Void in mineral aggregates (VMA)</td>
<td>SNI 03-6754-2002</td>
<td>16.8 16.8 16.9 16.9</td>
<td>%</td>
</tr>
<tr>
<td>Void fill bitumen (VFB)</td>
<td>SNI 03-6754-2002</td>
<td>74.8 74.8 73.1 73.1</td>
<td>%</td>
</tr>
<tr>
<td>Void in Mix (VIM)</td>
<td>SNI 03-6754-2002</td>
<td>4.3 4.3 4.6 4.6</td>
<td>%</td>
</tr>
<tr>
<td>VIM at Refusal Density</td>
<td>SNI 03-6754-2002</td>
<td>2.5 2.5 2.4 2.4</td>
<td>%</td>
</tr>
<tr>
<td>Stability</td>
<td>SNI 03-6758-2002</td>
<td>1063 1005 1286 1224</td>
<td>Kg</td>
</tr>
<tr>
<td>Flow</td>
<td>SNI 03-6758-2002</td>
<td>3.3 3.4 3.8 3.9</td>
<td>mm</td>
</tr>
<tr>
<td>Marshall quotient</td>
<td>SNI 03-6758-2002</td>
<td>322 296 338 314</td>
<td>Kg/mm</td>
</tr>
<tr>
<td>Retained stability after soaking</td>
<td>SNI 03-6753-2002</td>
<td>84.3 88.4 87.9 94.0</td>
<td>%</td>
</tr>
<tr>
<td>Tensile Strength Ratio (TSR)</td>
<td>AASHTO T 283 03</td>
<td>87.3 89.1 89.5 95.1</td>
<td>%</td>
</tr>
</tbody>
</table>

Figure 3. Marshall Stability and Marshall Quotient of the mixtures

Figure 4. Retained stability and TSR of the mixtures
6 CONCLUSIONS

Based on the data had been mentioned above, it could be concluded that Asbuton Pellets can increase mixture stability and also can act as an anti-stripping agent. The increment of retained stability and TSR by adding anti-stripping agent around 0.3% of asphalt is similar with the increment of retained stability and TSR by adding Asbuton Pellets around 2.5 % of the mixture.

7 ACKNOWLEDGEMENTS

I would like to thank to PT. Olah Bumi Mandiri for the supporting materials.

REFERENCES


**PAPER TITLE**  
The Accessibility of Paloh-Aruk Border Area at Sambas Regency West Borneo

<table>
<thead>
<tr>
<th>TRACK</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>AUTHOR</th>
<th>Position</th>
<th>Organization</th>
<th>Country</th>
</tr>
</thead>
<tbody>
<tr>
<td>Andrio Firstiana SUKMA, ST., M.Si.</td>
<td>Staff</td>
<td>Social, Economy and Environment Research Development Center, Ministry of Public Works</td>
<td>Indonesia</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CO-AUTHORS</th>
<th>Position</th>
<th>Organization</th>
<th>Country</th>
</tr>
</thead>
</table>

| E-MAIL (for correspondence) | | | |
|-----------------------------|------------------|
| sukma_af@yahoo.com | |

**KEYWORDS:**  
Border Area, Paloh-Aruk, Sambas Regency, Accessibility, Origin Destination Matrix

**ABSTRACTS:**  
Border area is one of the priority areas to develop in Indonesia. It was caused the change of paradigm since era of reformation that wants make border area as a front page with prosperity approach. One of border area that will be made as an activity center in the border of West Borneo and Sarawak-Malaysia according to Government Ordinance (Peraturan Pemerintah/PP) Number 26/2008 about National Spatial Planning (Rencana Tata Ruang Wilayah Nasional) is Paloh-Aruk National Strategic Activity Center (Pusat Kegiatan Strategis Nasional/PKSN). Administratively the area is in both Paloh and Sajingan Besar District at Regency of Sambas-West Borneo. Revitalization an existing road and construction a new road are some ways to realize that dream. This paper tries to formulate the accessibility of Paloh-Aruk Border. Based on analysis that done by forming Origin-Destination Matrix, it is known that movements in border area tend to happen in the village near the capital of regency than the border. This is due to limited infrastructure to the border. Moreover there is no movement to Malaysia because of the tight regulation after presence of custom and immigration office in both Indonesia and Malaysia.
The Accessibility of Paloh-Aruk Border Area at Sambas Regency West Borneo

Andrio Firstiana Sukma, ST., MSi.¹

¹Social, Economy and Environment Research Development Center, Ministry of Public Works, Surabaya, Indonesia
Email for correspondence: sukma_af@yahoo.com

1 INTRODUCTION

Border area nowadays become a priority area for development in Indonesia. It was caused of the switch of paradigm since reformation era that make border area as a front page with prosperity approach. It is also confirmed in Act No. 17/2007 about 2005-2025 Long-Term National Development Plan that shift the policy of border area from ‘inward looking’ to ‘outward looking’. Inward looking means the border area is the last area for being developed meanwhile outward looking means the border area is the key to develop a region so it can be used for a gateway of activity economic and trade with neighboring countries.

There are some regulations made to enhance the development in border area. One of them is Government Ordinance No. 26/2008 about National Spatial Planning that explain there are 26 border areas throughout Indonesia which will be developed by make a National Strategic Activity Center which contained urban areas with some buffer area to support the cities. It says that in 2019 all of 26 border areas are already fully developed and upgraded.

To achieve the target, it is important to know the existing condition of the border area such as condition of social Economic of the people, infrastructure that already built, potency and weakness of the area. Paloh-Aruk, which located in Sambas Regency, West Borneo, as one of the Border Area face a great a challenge to be one of the greatest front page. Inadequate infrastructure and public facilities, limited budget, high dependency on natural resources are the main problem to develop Paloh-Aruk Border.

Something has done to improve the condition. The priority is construct a new road and rehabilitate road. Also construct a bridge. In the Mountain area the pavement still soil pavement to prevent the damage from landslide. If there is a great river, raft or boat are used for crossing. All done to ease access from the city to border but all of it is still far from enough. The government and local authority still have many work to do.

However for all has done, the accessibility for Paloh-Aruk Border has increased. Unfortunately the existing accessibility is never calculated. Accessibility data both in Paloh-Aruk Border or Sambas Regency is not available. It is understandable because for border area the important thing are to build infrastructure needed and the accessibility will automatically follow. Moreover the infrastructure in Sarawak, Malaysia as neighboring country is much better than in Paloh-Aruk Border.

Based on explanation above, the purpose of the paper is to calculate accessibility in Paloh-Aruk Border Area. It can be used as baseline of Paloh-Aruk Border so we can know effect of everything done. The accessibility is counted by compose an origin destination matrix (OD Matrix). After the OD Matrix composed, we can know the most accessible areas in Paloh-Aruk Border.

2 BORDER AREA DEVELOPMENT

Borders serve 3 fundamental and broadly conceived functions (beside their basic role as the boundaries of state sovereignty): military (providing a barrier to military aggression from Broad); economic (constituting a barrier to the free movement of goods); and social (as a barrier to the free movement of people) (Komornicki, 2005). The significance of all three function began to change rapidly. At the beginning
the military function concentrated along the borders between different alliances. After the Second World War, the military function gradually decline and the borders became an economic and social functions.

It is possible to identify several basic phases to the functional changes along Europe’s State borders (Komornicki, 2005):

- Phase I. Maintenance of the significance of the military function of borders, with simultaneous development of the economic and social function – a situation that currently applies once more in the area of the former Yugoslavia
- Phase II. The decline of the military function with the maintenance of well-developed economic functions (customs) and social functions (restrictions on exist via passports and on entries via visas) – a situation which now hardly occurs along any European border
- Phase III. With this phase, the economic and social functions of borders are steadily limited. There is a liberalization of foreign trade and passport policy, with simultaneous retention of visa-mediated movements of people, and full border control over individual and goods (for example, to this day at The Norwegian-Russian Border). The transition to Phase III may be associated with the onset of economic integration
- Phase IV. The trade in goods undergoes further liberalization (usually as a consequence of economic integration). Non-visa travel is introduced, though border controls over people and goods are retained (as along the Polish-German border until May 2004)
- Phase V. There is a full liberalization of the trade in goods (lifting of the majority of custom duties and fees at borders). Visa-free travel gives way to full freedom of movement, the taking-up of work and changes in place of residence. Border controls over people and goods are simplified and minimized (as at present along the French-Swiss Border)
- Phase VI. The elimination of the border Control, thereby permitting the crossing of borders at any point (as in the case of the Schengen Group of countries or between Belgium-France from 1991)

Development in border area have similarities with other types of regional development. There are 4 similarities of both development namely: the role of infrastructure; the significance of transportation cost; the importance of factor supplies and the crucial role of government in promoting development (Wu, 2001). However there are uniqueness about cross-border development. Wu (2001) explain 6 characteristics of cross-border development:

- **The Immobility of factors of production.** Conventional development assume that there are no obstacles to the mobility of factors of production but in the cross-border development the border itself is an important barrier that immobilizes factors of production mostly the movement of people
- **Transaction costs and delays.** Borders impose their own transactions costs in the form of delays caused by clearing customs, traffic congestions, and other bureaucratic impositions
- **Incompatible economic systems.** Some of the more challenging cases of cross-border development occur at borders where a transitional economy and a Market-oriented economy meet. Cross-border developments that involve 2 or more transitional economies at different stages of evolution tend to entail additional complications
- **Institutional issues.** Institutional issues such as profit repatriation regulations, double taxation agreements, guarantees of exchange rates and political institutions for joint actions are crucial to cross-border developments
- **Proximity of differences.** The economic complementary of 2 territories involved in cross-border development might best understood as the proximity of differences. This is especially evident in cases of contiguous territories.
• **The role of informal sector.** Many cross-border developments are based on informal sectors activities, including trade and small-scale industries. Informal sector activities are particularly germane in cases of spontaneous development that evolve from trade.

3 ACCESSIBILITY AND ORIGIN DESTINATION MATRIX

Accessibility is a measure of convenience or ease about how nodes interact each other and 'easiness' or 'difficulty' to reach nodes by transportation system (Tamim, 2000). Now the question is, what is the standard of easy and difficult because very subjective. Therefore the measurable performance is needed to express accessibility. Distance is one of measures for accessibility. If 2 nodes are near each other, we can say that accessibility between both nodes are high. Another measure is location (Rodrigue, Comtois, & Slack, 2006). The nearest nodes to transport facilities have the most accessibility. In the end, travel time are the most common measure to count accessibility (Tamim, 2000). So in this paper, I choose travel time as a proxy measurement of accessibility.

After determining measure of accessibility, the next step is count flow in the desired area. There are 3 types of flow: vehicles, goods and passengers. The more flows, the area is more accessible. We can count flows by compose an OD Matrix. The area is divided into zone and OD matrix illustrate flows in the area per zone. OD matrix consist of row that show origin zone and column that show destination zone so the cell show the flow from origin zone to destination zone. There are many methods to fill cell, one of them are gravity model (GR).

4 THE STUDY AREA

Paloh-Aruk Border Area is located Sambas Regency, West Borneo. Administratively, Paloh-Aruk consists of 2 districts which is Paloh and Sajingan Besar. Paloh is located north of Sambas with a distance 49 km meanwhile Sajingan Besar located northeast of Sambas with a distance 83 km. There are 13 villages in Paloh-Aruk where 8 in Paloh and 5 in Sajingan Besar. From 13 villages, 7 are directly land bordering with Malaysia.

Total area of Paloh-Aruk are 254,004 ha. This is 39.71% from Sambas Regency and make Paloh and Sajingan the widest districts in Sambas Regency. The widest village is Sungai Bening in Sajingan Besar Districts with the area is 55,730 ha or equal with 22% area of Paloh-Aruk meanwhile the smallest village is Matang Danau in Paloh Districts with the area only 6,487 ha or 2.55% area of Paloh-Aruk.

There are already road network that connect border area in Paloh and Sajingan Besar to Sambas but the condition is really horrible especially in Sajingan Besar. IRI data showed that most road in Paloh-Aruk is above 10 which means heavily damaged and can only be crossed by a double-axle vehicle. In addition, many bridges needed because the existing bridges are in poor condition and in some great rivers there are no bridges.

Nowadays an official custom and immigration office has been opened in Aruk. Administratively, Aruk is located in Sebunga Village, Sajingan Besar Districts. Although the office is already operated but the activities is not too busy. Terrible access to the border maybe the cause even though the distance to Kuching, Sarawak-Malaysia is close.
5 METHODOLOGY

The methodology provides a procedural outline used for the conduct of this research work. As mention above, the objectives of the research are to analyze accessibility in Paloh-Aruk Border Area by compose an OD matrix with gravity model. The assumption in gravity model is trip production and trip attraction linked with parameter in origin and destination zone also linked with accessibility. Measure used for accessibility in this paper is travel time. The equation of GR model is

\[ T_{id} = K \frac{O_i \cdot d_{id}}{d_{id}} \]  

(1)

Which,

\- \( T_{id} \) = flows from origin zone (i) to destination zone (d)
\- \( K \) = Constanta
\- \( O_i \) = value of trip production in origin zone (i)
\- \( D_d \) = value of trip attraction in destination zone (d)
\- \( d_{id} \) = travel time from origin zone (i) to destination zone (d)

Step for composing OD Matrix with GR model as follows,
1. Paloh-Aruk Border Area is divided into 13 zones where every zone is every village in 2 districts plus 6 external zones which 3 zones are districts that directly bordering, a zone is Sambas as a capital and 2 zones are in Malaysia as they directly bordering.

Table 1 Zones Division

<table>
<thead>
<tr>
<th>Districts/Villages/Zones</th>
<th>Center of Zones</th>
<th>No. of Zones</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Paloh</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kalimantan</td>
<td>Kalimantan</td>
<td>1</td>
</tr>
<tr>
<td>Matang Danau</td>
<td>Matang Danau</td>
<td>2</td>
</tr>
<tr>
<td>Tanah Hitam</td>
<td>Tanah Hitam</td>
<td>3</td>
</tr>
<tr>
<td>Mentibar</td>
<td>Mentibar</td>
<td>4</td>
</tr>
<tr>
<td>Malek</td>
<td>Malek</td>
<td>5</td>
</tr>
<tr>
<td>Nibung</td>
<td>Liku</td>
<td>6</td>
</tr>
<tr>
<td>Sebubus</td>
<td>Merbau</td>
<td>7</td>
</tr>
<tr>
<td>Temajuk</td>
<td>Temajuk</td>
<td>8</td>
</tr>
<tr>
<td><strong>Sajingan</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sentaban</td>
<td>Sasak</td>
<td>9</td>
</tr>
<tr>
<td>Senatab</td>
<td>Sp. Tanjung</td>
<td>10</td>
</tr>
<tr>
<td>Kaliau</td>
<td>Keranji</td>
<td>11</td>
</tr>
<tr>
<td>Sebunga</td>
<td>Aruk</td>
<td>12</td>
</tr>
<tr>
<td>Sei Bening</td>
<td>Sei Bening</td>
<td>13</td>
</tr>
<tr>
<td><strong>Tangaran</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Simpang Empat</td>
<td>14</td>
</tr>
<tr>
<td><strong>Teluk Keramat</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Sekurai</td>
<td>15</td>
</tr>
<tr>
<td><strong>Galing</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Galing</td>
<td>16</td>
</tr>
<tr>
<td><strong>Sambas</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Sambas</td>
<td>17</td>
</tr>
<tr>
<td><strong>Lundu District, Malaysia</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Lundu</td>
<td>18</td>
</tr>
<tr>
<td><strong>Kampung Teluk Melano, Malaysia</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Teluk Melano</td>
<td>19</td>
</tr>
</tbody>
</table>

Source: Analysis Results (2013)

2. Since the measure of accessibility is time travel, so the next step is counted travel time between zones

3. The next step is count potential flows between zones using socio-economic variable. This paper used gross regional domestic product (PDRB) with consideration PDRB is the most appropriate parameter to show economy in a region

4. After have potential flows now we count the actual flows by count the traffic. Primary survey is used in this paper to get traffic data. There are 6 spots where traffic counting done namely Sp. Tanah Hitam, Liku, Merbau, Sp. Sasak, Sp. Tanjung and Aruk Border
5. The potential flows obtained before then must be plotted in the road map. This is called trip distribution. Every potential flows between zones are distributed to road segments existing.

6. After all potential flows plotted in the road segments, next step are compared between actual flows with potential flows that already plotted in the road segments. By doing regression analysis with iteration we will find Constanta K and power n in equation (1)

7. With K and n is known, now we can compose OD Matrix and finally we will know the accessibility of Paloh-Aruk Border Area and also zone which has the highest accessibility

6 RESULTS AND DISCUSSION

Using the methods explained above, this is GR Model for Paloh-Aruk Border Area,

\[ T_{id} = 5.10^{-19} \frac{O_t \cdot D_d}{d_{id}^{17}} \]

After we know the equation, we can compose OD matrix for Paloh-Aruk Border Area,
Table 2: OD matrix Pekanbaru—Bintan Area

<table>
<thead>
<tr>
<th>Source Analysis Results (2013)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1178 2,093.7077 655.1510 6.6123 20.2137 36.9571 25.3011 340.8563</td>
</tr>
<tr>
<td>Kab. Sambas 317.2062</td>
</tr>
<tr>
<td>Kawasan PALSA 97.1183</td>
</tr>
<tr>
<td>-</td>
</tr>
</tbody>
</table>
Every cell in the OD matrix showed flows from origin zone to destination zone using PDRB as the push and pull factors. This is the limitations of study that only using only 1 variabel. However, we can see from the OD Matrix above that the most accessible zone is zone 15 (Teluk Keramat District). Although its PDRB is not too high, but with its location that became intersection point between Sambas and both border area: Sajingan Besar and Paloh, make Sekurai (center of Teluk Keramat District) the most accessible place.

Also from the OD matrix above, we can see most of village that far from Teluk Keramat District have worst accessible. It is easily understand because beside of the distance, the road condition to those villages are really horrible meanwhile it is not easy to go to Serawak, Malaysia due to tight regulation after presence of official custom and immigration office.

We also can see that the accessibility in Paloh Border is better than in Sajingan Besar Border. Zone 1-8 (Paloh Border) have better accessibility than zone 9-13 (Sajingan Besar Border). This happen perhaps Paloh Border produced more resources needed in daily life such as rice and fish than in Sajingan Besar Border that still initiated palm plantations and need time to grow.

The other factor that make Paloh Border more accessible than Sajingan Besar border are the natural condition. Paloh has more great soil condition than Sajingan Besar. The slope in Paloh also more gentle than in Sajingan Besar. Moreover people of Paloh mostly Malays who prefer live in groups and stay near the road meanwhile Sajingan Besar mostly Dayak that prefer live exclusively in the forest.

7 CONCLUSIONS

Although there are road that connected border area with Sambas, but with the terrible conditions make travel from the border to Sambas or vice versa obstructed. Consequently, flows of traffic tend to happen near to Sambas and goods are not distributed to border area make people in border fulfill their needs by buying product of Malaysia. But in the other hand, with the official custom and immigration office, flow of goods is delayed and increase costs.

For solving that problem, there is no other ways but to improve the quality of road. If the budget is limited, it can be prioritized to Sajingan Besar District that has bad accessibility. Also with great accessibility of Sekurai Districts, it can be used to focusing the economy in Sekurai. At least this will make travel time of people in border area faster than they have to go to Sambas.

BIBLIOGRAPHY


# Mitigating Traffic Congestion in Asia: A comparative analysis

<table>
<thead>
<tr>
<th>TRACK</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>AUTHOR (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dr. Jonathan Luke Sturtz</td>
<td>Global Product Manager</td>
<td>Lindsay Transportation Solutions</td>
<td>USA</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CO-AUTHOR(S) (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
</table>

| E-MAIL (for correspondence) | Jon.sturtz@Lindsay.com |

**KEYWORDS:**

1. Managed Lanes
2. Congestion Management
3. Traffic Congestion
4. Concrete Barrier
5. Moveable Barrier
Mitigating Traffic Congestion in Asia: A Comparative Analysis

Dr. Jonathan Luke Sturtz

1Lindsay Transportation Solutions, Omaha, Nebraska, USA
Email for correspondence: jon.sturtz@lindsay.com

1 INTRODUCTION TO THE TRAFFIC CONGESTION PROBLEM

Governments and road operators in Asia face challenges associated with traffic congestion. As a region that is quickly growing, from both a population and economic perspective, governments and road operators must contend with traffic congestion issues. As an example of the growth, Asia adds 120,000 people each day; moreover, this dramatic population growth is going to make traffic congestion even worse in the coming years because the number of vehicles on the road in Asia doubles every five to seven years (Urban transport, 2014).

Traffic congestion costs governments, organisations and individuals in Asia. For example, in Jakarta, Indonesia, data show that traffic congestion costs the city up to 46 trillion rupiah (5.2 billion USD) per year (Arditya, 2011). The situation is similar across Asia, where studies indicate that congestion costs Asia between 20 billion USD to 50 billion USD every year “Urban transport,” 2014). The costs are not just economic; they are also environmental. According to “Urban transport” (2014), Asian cities suffer from the worst pollution in the world and traffic congestion is responsible for some of this pollution. From both economic and environmental perspectives, individuals, organisations and governments in Asia pay for traffic congestion.

To address traffic congestion, road operators often consider building additional roadway capacity. The benefit of increasing the number of lanes or roadways in highly congested areas is clear. More lanes or alternative roadways would mitigate some of the traffic congestion. The problem with this approach to mitigating congestion is that it is quite expensive. Even though additional roadway capacity would address traffic congestion in Asia, cost is often a barrier to implementing this solution.

2 CHALLENGES ASSOCIATED WITH BUILDING ADDITIONAL ROADWAY CAPACITY

One of the drivers of construction cost is the specific conditions. When the road construction application involves either tunnels or bridges, then the construction cost is even higher than other construction types. For instance, the cost to build a tunnel ranges from 20 million USD to 80 million USD per lane kilometre (Rostami et al., 2013). The cost of building roadways with bridges or elevated roadways on them is also quite high. The “Construction and rehabilitation cost guide” (2010), indicated that the cost of building elevated roadways with three lanes in each direction ranges from 18 million USD to 26.66 million USD per lane kilometre. Building additional roadway capacity is sometimes prohibitively expensive and the cost certain types of construction, such as tunnels and elevated roadways are higher than other conditions.

Direct construction factors are a major contributor to the expense associated with building additional roadways that have either tunnels or elevated portions on them. Direct construction factors include geology, excavation type, materials/plant, end use, length, face area, depth/height, lining type, location, local labour cost (Efron & Read, 2012; Rostami, et al., 2013). Geology is the nature of the specific earth and other materials through which the tunnel must be constructed or the on which the elevated roadway must be built. The excavation type is the digging method the geology and other factors require.

The composition of the tunnel or elevated roadway is a major determinate of the materials. The expected end use captures the anticipated ways in which the roadway will be used. The length simply refers to the overall distance of tunnel or elevated roadway, while the face area is a tunnel-specific concept to describe the area of the outside of the tunnel. Depth refers to how deep engineers design the tunnel, while height is an elevated roadway term that refers to how high engineers design an elevated roadway.
Lining type is a tunnel-specific term that describes that materials that line the tunnel. The location captures the relative difficulty associated with site access and local labour cost describes the expense associated with obtaining local workers for the project. Direct construction factors are a significant driver of the cost and complexity associated with building additional roadway capacity.

In addition to the direct construction factors faced by road authorities, when building additional roadway capacity, road owners also must contend with direct costs. Some of the factors that influence direct tunnel or elevated roadway costs include: land costs, span, environmental constraints, project size, aesthetic issues, weather, location, skew, pile footing, abutment, construction stages (Archer & Glaister, 2006; Pugh et al., 2012).

Land cost refers to the expense associated with purchasing the appropriate right of way on which to build the project. Span is an elevated roadway-specific term that refers to the distance from one end of the elevated roadway to another (or from one support to another). Environmental constraints capture the site-specific complexity associated with properly minimizing the project’s negative environmental effects. Project size simply encompasses the concept that project size influences project cost beyond the obvious cost multipliers (e.g., labor and materials). Construction stages describe the ways in which construction stages influence project cost.

Aesthetic issues describes the relationship between architectural issues and project cost, while weather refers to the influence that weather conditions have on project cost. The location of the project also has a direct bearing on project cost. An elevated roadway-specific term is skew, which refers to the curvature of the bridge. Pile footing describes the costs associated with securing the elevated roadway to the ground at the structure’s midpoints, while abutment refers to connecting the elevated roadway at its end points. Road authorities seeking to build additional roadway capacity as a means to mitigate traffic congestion face substantial direct costs.

In addition to both the direct cost factors associated with building additional roadway capacity, road authorities must also consider both micro and macro factors associated with road construction. Micro factors are those that relate directly to the project. These include: time spent by various stakeholders, the construction’s effect on surrounding organisations and road user costs (Goodrum et al., 2006). Road authorities building additional capacity even face macro cost factors, which are those elements that would affect any infrastructure project. Health and safety, government support, regulatory environment and market competition are among these macro cost factors (Efron & Ready, 2012). To build additional roadway capacity, road authorities face both micro and macro construction cost factors.

Road authorities often consider building additional roadway capacity as a means to mitigate traffic congestion; however, the authorities face challenges with expanding the road network from a number of dimensions. The specific construction factors are relevant. Additionally, both direct and indirect cost factors present barriers for building additional road capacity.

3 USING MANAGED LANES TO ADDRESS TRAFFIC CONGESTION

Because of the aforementioned challenges associated with building additional roadway capacity, as well as a number of other factors, road authorities have increasingly turned to managed lanes as an important part of the overall approach to mitigate congestion. Kuhn et al. (2005) indicate that six main managed lane strategies exist: HOV lanes, HOT lanes, exclusive lanes, mixed flow separation/bypass lanes, lane restrictions and dual facilities. Road authorities use one or more managed lanes approach as a means to mitigate congestion within the road network.

Two of the most common managed lane facilities are high occupancy vehicle lanes (HOV lanes) and high occupancy toll lanes (HOT lanes). Konishi and Ko (2009) indicated that HOV and HOT lanes are lanes that carry vehicles with some predetermined number of people; however, the difference between HOV and HOT lanes is that drivers with fewer than the number of required passengers to travel in the lane may use
the lane for a fee. A variation of HOT lanes is variable pricing, or even usage, that is based on traffic conditions (Kuhn et al., 2005).

Separated, concurrent flow and contraflow HOV and HOT lanes are typical approaches to a managed lane strategy; moreover, physically separating the managed lanes from the general purpose lanes is the most effective method to ensure successful system use. Kuhn et al., (2005) indicate that right-of-way issues as well as many other topics often limit choices with respect to implementing managed lanes, which requires more strategic use of road capacity. According to “NCFRP report 3: Separation of vehicles: CMV only lanes, 2010” exclusive lanes, mixed flow/bypass lanes, lane restrictions and dual lane facilities are all methods of controlling or separating different types of vehicles.

Road authorities often physically separate managed lanes from general purpose lanes because this separation of lanes is the most effective method to deploy and operate managed lane facilities; however, each of the methods to physically separate managed lanes has both advantages and disadvantages. When managed lanes involve physical separation, four different types of approaches exist: those that separate the managed lanes from the general purpose ones through the use of different grades, those that separate the managed lanes from the general-purpose lanes through the use of traffic control devices, those that separate the managed lanes from the general-purpose lanes through the use of permanent positive protection and those that separate the managed lanes from the general-purpose lanes through the use of moveable positive protection (“Pricing managed lane guide, 2013).

Hlavacek et al. (2007) and Drăgan (2013) posited that road authorities must consider cost, safety and compliance when they implement and operate managed lanes. The taxonomy in Figure 1 outlines the relationship between these three elements. Cost is the expense associated with implementing the solution. Safety refers to the protection the managed lane strategy provides to the users. Compliance relates to the relative difficulty that road operators face with keeping road users in their designated lanes.
When considering the four approaches physically separated managed lanes (grade, traffic control devices, permanent positive protection and moveable positive protection), road authorities should evaluate each along cost, safety and compliance considerations. The first approach, using dual grade, has high safety and compliance effectiveness; however, the approach requires limited use lanes in the sense that the lanes are only for high occupancy vehicle traffic. Because of this, grade separated managed lanes have low cost effectiveness to mitigate traffic congestion. The second approach to physically separated lanes, using traffic control devices has excellent cost benefits. At the same time, traffic control device separated managed lanes, in developing regions such as Asia, do very little to address either safety or compliance. For this reason, the use of traffic control devices to physically separate managed lanes has low safety/compliance effectiveness.

Permanent barrier protection is the third method to physically separate managed lanes. While this approach has high safety and compliance effectiveness because of the separation between the managed general purpose lanes, the method also has low cost effectiveness because of the limited use associated with permanently separated managed and general purpose lanes. Moveable concrete barrier is the fourth method to physically separate managed lanes. The approach provides both high safety/compliance and high cost effectiveness. Moveable concrete barrier offers high safety/compliance because it separates the managed lanes from the general-purpose lanes through the use of physical separation. Additionally, moveable concrete barrier is highly cost effective because it can apply unused roadway capacity on the free flowing direction of the roadway to the congested direction of the roadway.

<table>
<thead>
<tr>
<th>Cost Effectiveness</th>
<th>Safety/Compliance Effectiveness</th>
</tr>
</thead>
<tbody>
<tr>
<td>High Cost</td>
<td>Both High Cost Effectiveness and High Safety/Compliance Effectiveness</td>
</tr>
<tr>
<td>Both Low Cost</td>
<td>Low Cost Effectiveness but High Safety/Compliance Effectiveness</td>
</tr>
<tr>
<td>Low</td>
<td>Both Low Cost Effectiveness and Low Safety/Compliance Effectiveness</td>
</tr>
</tbody>
</table>

Figure 1. Taxonomy to evaluate competing managed lane solutions.
According to Eng (2011), movable concrete barriers are those barriers that can quickly and easily be moved 1 2 metres to 5 4 metres using a barrier transfer machine. Movable concrete barrier provides road operators the ability to quickly and easily reallocate travel lanes for managed lanes in the peak traffic direction (Eng, 2011).

Because of the high cost and other issues associated with building additional roadway capacity, road authorities are increasingly turning to managed lanes as a traffic congestion mitigation strategy. Often these managed lanes involve physical separation. Of the methods to physically separate managed lanes, moveable concrete barrier is the ideal solution that offers both high cost effectiveness and high safety/compliance effectiveness.

4 AN EXAMPLE OF MOVEABLE CONCRETE BARRIER

While physically separated managed lanes provide clear benefits to road authorities, they can also present some challenges. Kuhn et al., (2005) indicated that managed lanes often cause movement from general purpose lanes to managed lanes, which results in increased accidents. This is particularly true if traffic control devices are the only method used to separate the managed and general purpose lanes. A second issue associated with managed lanes is compliance (Drăgan, 2013). When traffic control devices are the only method to separate managed and general purpose lanes, road authorities face the challenge of collecting tolls (if a toll facility) and road operators face the chaos associated with users staying in designated lanes.

Permanent concrete barriers provide an effective means to address the safety and compliance topics “NCFRP report 3: Separation of vehicles: CMV only lanes, 2010.” The problem with concrete barriers is that they drive up cost because they do not effectively use available roadway capacity (Kuhn, 2005). Moveable concrete barrier addresses the safety topic by providing positive separation between the managed lanes and the general purpose lanes. Moveable concrete barrier can also addresses incremental cost issue by eliminating the need for two or more lanes (through more effective use of existing lanes).

Because road authorities cannot fund building additional roadway capacity, they often cancel proposed projects. Using moveable concrete barrier to create managed lane facilities can be the solution that makes road projects viable because this approach stretches infrastructure funding. The Columbia River Crossing is an example of a cancelled project that could have been viable with moveable concrete barrier.

According to “What are the problems? (2014), the Columbia River Bridge is bridge in the United States that moves 134 000 vehicles per day from Washington state into the Portland metropolitan area. The project was conceived to mitigate the four to six hours of daily traffic congestion and to improve the roadway’s accident rate that was double the rate of typical urban freeways (What are the problems?, 2014). The project was canceled in May 2014 due to a lack of funding (What are the problems?, 2014).

Moveable barrier would have eliminated the need for two lanes of this bridge. As seen in Table 1, with a cost of capital of 4%, 30 years to payback, a construction cost 28 million USD and an annual roadway maintenance cost of 45,430 per lane kilometre USD, the lifetime cost of the two lanes of the project would have been ~45 million USD.

Moveable barrier might have saved this project. Table 1 shows that a cost of capital of 4%, 30 years to payback, an implementation cost of 5.5 million USD and an annual operation cost of 63 541 USD per lane kilometre, the lifetime cost of moveable barrier for the project would have been 14 million USD. Moveable barrier would have saved the project ~31 million USD and may have been enough of a cost savings to move the project forward.
Table 1. Cost Comparison of movable barrier and building additional capacity

<table>
<thead>
<tr>
<th>Expense</th>
<th>Build Two Extra Lanes</th>
<th>Use Moveable Barrier for Two Lanes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Capital</td>
<td>~28 000 000 USD</td>
<td>~5 500 000 USD</td>
</tr>
<tr>
<td>20 year Operating Cost</td>
<td>N/A</td>
<td>~6 1 million USD</td>
</tr>
<tr>
<td>Roadway Maintenance</td>
<td>~4 3 million USD</td>
<td>N/A</td>
</tr>
<tr>
<td>Cost of Capital</td>
<td>~12 7 million USD at 4%</td>
<td>~2 4 million USD at 4%</td>
</tr>
<tr>
<td>Total 30 Year Cost</td>
<td>~45 million USD</td>
<td>~14 million USD</td>
</tr>
<tr>
<td>Total Annual Cost</td>
<td>~2 25 million USD</td>
<td>~0 7 million USD</td>
</tr>
</tbody>
</table>

Note: Adapted from Pugh et al. (2012); Rostami et al. (2013); Sinha et al. (2005)

4 CONCLUSION

Faced with rapid economic and population growth, governments across Asia contend with ever escalating congestion. Often, building additional roadway capacity as a solitary traffic mitigation solution is impractical because of the associated cost of this approach. As a result, road authorities turn toward managed lanes as another traffic mitigation strategy. To implement managed lanes, road authorities must balance cost, safety and compliance. Moveable concrete barrier is a managed lane strategy that offers both high safety and high cost effectiveness. This approach is can be a good way to stretch infrastructure funding and may increase the number of traffic mitigation projects that road authorities can undertake.
REFERENCES


Garrett, M. (2014, March 7). ODOT announces plans to close down Columbia River Crossing Project. Oregon Department of Transportation


Hlavacek, I., Vitek, M., & Machemehl, R. (2007). Best practices: Separation devices between toll lanes and free lanes. Center for Transportation Research at The University of Texas at Austin


The most important aspects to select a measurement device for pavement bearing capacity survey

José Antonio Ramos-García¹
Advanced Technologies Director
Euroconsult Group
Avenida Montes de Oca 9-11, Parque Empresarial Sur
28700 San Sebastián de los Reyes – Madrid (Spain)
Tel.+34 916 59 78 31 Fax.+34 916 59 78 10
Email: jramosg@euroconsult-group.com

Fernando Sánchez-Domínguez
Innovation Director
Euroconsult Group
Avenida Montes de Oca 9-11, Parque Empresarial Sur
28700 San Sebastián de los Reyes – Madrid (Spain)
Tel.+34 916 59 78 31 Fax.+34 916 59 78 10
Email: fsanchezd@euroconsult-group.com

KEYWORDS
Pavement deflection, Curviometer, Road network management, Project level, Construction level.

ABSTRACT

The bearing capacity of a road pavement depends, essentially, on the deflection, the thickness of each layer, the type of materials used (subgrade and pavement structure) and their condition.

Non-destructive deflection testing is one of the most reliable methods available to determine the structural condition of an in-service pavement. The measured pavement deflection is used to determine the structural adequacy of a pavement, related to the number of allowable load repetitions.

The idea of bearing capacity is not solely limited to the overall behavior of a road’s pavement, but it must be considered on each of the layers that make up the pavement, from its subgrade to the wearing course layer. That’s why, it is a concept that arises during the project stage and that includes all the constructive stages.

This document analyzes the different fields of application of the pavement deflections: Road network management, Project level and Construction control.

Depending on the field of application, the deflection survey needs could be different. This paper studies the most important aspects to be taken into account when a measurement deflection is planned. Some of these aspects are the following: Measurement speed, Sampling rate and report and Type of results (maximum deflection, entire bowl deflection, among others) and precision.

¹ Corresponding author
The most important aspects to select a measurement device for pavement bearing capacity survey

José Antonio Ramos García & Fernando Sánchez Domínguez
Euroconsult Group, San Sebastián de los Reyes, Madrid, Spain
jramosg@euroconsult-group.com; fsanchezd@euroconsult-group.com

1. INTRODUCTION

The bearing capacity of a pavement determines its residual life and the need for maintenance or rehabilitation for a specific traffic volume over a certain period of time. It is not currently possible to directly measure the bearing capacity of a road. However, it can be mainly determined by the entire deflection bowl recorded for an established load.

This concept is indeed linked to in-service roads management and pavement rehabilitation projects. Nevertheless, the idea of bearing capacity is not solely limited to the overall behavior of a road’s pavement, but it must also be considered on each of the layers that make up the pavement, from its subgrade to the wearing course layer. That is, it is a concept that appears during the project stage and includes all the constructive stages. This paper shows the main devices aimed at assessing pavement and subgrade bearing capacity on execution and maintenance condition control nowadays.

2. FIELDS OF APPLICATION

Pavement characteristic parameters surveys (as it is the case of the entire deflection bowl) have been traditionally linked to maintenance. In order to know the pavement condition and be able to undertake any rehabilitation plan, it is necessary to have information about the different parameters that determine its behavior.

Action Cost 324 (European Commission 1997a) states that, when performing the survey of a network – always keeping in mind the possibility of developing analytic models that allow to assess the pavement’s behavior – it is necessary to carry out automated inspections of the network’s condition that will at least measure any deflections, wearing course appearance longitudinal profile, macrotexture and skid resistance. These parameters are the basis of any maintenance action.

Regarding deflections, their measuring must be performed as it is shown in Action Cost 325 (European Commission 1997b) in order to achieve different goals, like the following (among others): to research the need for rehabilitation, to obtain the stiffness modulus from the different pavement layers, to calculate the pavement’s residual life, to evaluate structural capacity, to identify areas with a worse behavior, to establish priorities regarding road rehabilitation, to survey every layer of the pavement during the construction stage, to plan maintenance and to do new research.

Both documents (European Commission 1997a and European commission 1997b) clearly state the importance of having information about pavement condition parameters for their maintenance. In addition, Action Cost 325 (European Commission 1997b) also includes the importance of surveying every layer of the pavement during the construction stage.

Along these lines, the possibility of linking construction with maintenance management seems also obvious, since the data measured during the construction stage will serve as the maintenance “zero point”. In short, the application horizon of a parameter survey to assess the bearing capacity of a pavement could be summarized as follows: Construction process, Connection between construction and maintenance “zero point” and Maintenance management.

3. MAINTENANCE MANAGEMENT

This section presents the main deflection measuring devices, the processing of the obtained data, and some application examples of maintenance management.
3.1. Deflection Survey Devices

In a nutshell, Table 1 shows the main deflection survey devices currently used. This table only includes systems that measure maximum deflection and the entire deflection bowl. There are different published correlation studies among these systems. Besides, there are other devices –like high performance laser deflectographs– that are being assessed by different national administrations and laboratories to use them at network level; the first stages of the assessment are yielding promising results.

Table 1. Deflection Measuring Systems (source: Benatov and Sánchez, 2007)

<table>
<thead>
<tr>
<th>MEASURING SYSTEM</th>
<th>TEST STANDARD (*)</th>
<th>MEASURING SPEED</th>
</tr>
</thead>
<tbody>
<tr>
<td>Falling Weight Deflectometer</td>
<td>NLT-338 (2007)</td>
<td>Punctual device</td>
</tr>
<tr>
<td>LaCroix Deflectograph</td>
<td>NLT-337 (1992)</td>
<td>2-4km/h</td>
</tr>
<tr>
<td>Curviameter</td>
<td>NLT-333 (2006)</td>
<td>18km/h</td>
</tr>
</tbody>
</table>

(*) These devices are included in different international standards (not only in the Spanish regulations cited in this table)

In the case of a vast road network, it would be convenient to determine the deflection with a device that combines the speed during data acquisition and the possibility of recording the entire deflection bowl, in order to analyze the pavement behavior at a project level. The features of the curviameter device make it the most suitable system to measure deflection in a road network with respect to a Pavement Management System (Ramos et al., 2013).

3.2. Curviameter System

Curviameter test standards and documents [Spanish NLT-333 (CEDEX 2006), French NPF-98-200-7 (AFNOR 1991), German FGSV 433 B 4 (FGSV 2011), Belgian 54.26 (MINISTÈRE DE LA RÉGION WALLONNE 2002), among others] include a detailed description of the measuring system and the test preparation, the sensors calibration, as well as the measurement principle and procedure.

![Figure 1. Overall diagram of the curviameter device (source: NLT-333, CEDEX 2006)](image)

The curviameter is a measuring system (Figure 2) consisting of a chain that, thanks to the necessary mechanisms, rotates in a synchronized way with the truck on which it is installed. Said truck has two axes. Its back axis -a simple axis with twin wheels- is applied a load that can be adjusted between 80 and 130kN, following the standard of each country. This chain has three measuring sensors -located between the twin wheels-. The chain is 15m long with a separation of 5m between each sensor. The data acquisition is performed at a speed of 5m/s (18km/h) and is repeated every 5m. The entire deflection bowl is obtained from the signs provided by the sensors (geophones) and is defined by 100 points on a length of 4m.

3.3. Adjustment Factors

The obtained deflections depend on the measuring conditions, that is, they vary depending on the temperature of the bituminous mixes and subgrade moisture during the data collection. It is then convenient to apply adjustment coefficients to the measured deflections. Broadly speaking, each country’s regulations reflect the theoretical-empirical coefficients that must be applied on each case, adapted to the materials and weather of each area.
3.4. Statistical Analysis Of The Obtained Values

Once the measurements are corrected, the data are statistically analyzed building on the large volume of information, which allows to define in detail any sections with homogeneous behavior.

After establishing homogeneous areas, each of them is analyzed separately and a characteristic deflection is then calculated (by using a specific confidence level).

3.5. Assessment Of The Obtained Results

When analyzing the surveyed deflections, there are different methods of interpretation so as to determine the condition in which road pavements are found and which would be the suitable rehabilitation actions.

Some rehabilitation standards are based on the analysis of maximum deflections, both punctual ones and those representative of homogeneous areas, as happens with Spanish Standard 6.3-IC (Spanish Ministry of Development, 2003). It includes the minimum reinforcement thickness with bituminous mixes suitable for each section, function of the representative deflection, traffic and type of pavement section.

Other standards and recommendations, such as the AASHTO Pavement design guide (AASHTO 1993), use the deflections surveyed on the pavement in order to determine -together with the rest of necessary parameters- the resilient modulus of the subgrade and to assess the structural condition of the current pavement and its rehabilitation needs.

There are also methods based on analytical models in which the characteristic moduli of the materials that form the pavement are defined through back-calculations from the deflection bowl and information about the pavement structure. With this information the pavement is modelled, and stresses and strains of every layer are analyzed, simulating the effect of the action of a vertical load uniformly distributed on the pavement. From the stresses and strains obtained with the response model, the residual life of the pavement is studied thanks to fatigue laws, which allow to design the appropriate rehabilitation solutions. In general terms, inverse calculation software [for instance, Elmod software (Henriksen 2006)] are designed to evaluate deflections recorded by a falling weight deflectometer. Nevertheless, there are also inverse calculation software that have been specifically designed for deflections surveyed by the curvimeter, as it is the case of DimMet software (Maecck 2009), developed by the Belgian Road Research Centre (BRRC).

Moreover, it should also be stressed that the application of correlation coefficients among the deflections recorded by the different measuring systems at different distances from the load application point, enables the standardization of deflections, as well as its use in the different inverse calculation software, regardless of the measuring device being used.
On the other hand, there are many complementary studies that aim to relate the measurements obtained directly by the curviameter to the characteristic pavement parameters and its residual life. Among them, the one Gorski established (Gorski 2005) must definitely be highlighted. It showed the relationship between the deflection recorded by the curviameter when the load is 900mm away from the measuring sensor and the representative modulus of the pavement subgrade. In turn, the BRRC established a relationship (BRRC 1998) between the characteristic deflection ($d_k$) obtained by the curviameter (applying a load of 100kN) and the number of equivalent axes of 80kN.

![Figure 3. Usage of curviameter data in a network](image)

Lastly, the existence of other studies mainly developed in France that deal with the interpretation of the curvature radius product and the curviameter maximum deflection must also be mentioned, as well as the relationship between the curviameter curvature radius and the adherence between layers, establishing a division of classes (Kobisch 2008 and Chea 2006).

### 3.6. Examples

The curviameter has been used on a regular basis to survey road networks in different European countries, like Spain, France, Portugal and Belgium for a long time. These devices are capable of measuring thousands of kilometers per year, with the aim of obtaining information of a network for its management (Pavement Management Systems input data), thus acquiring a parameter that is sufficiently precise to achieve the level status of a project. Moreover, there has been an international repercussion reaching other countries, both European (Germany and Poland among others), and American (like Mexico and Brazil).

Figures 3 and 4 show two application examples. The first one (Figure 3) corresponds to a network’s management, whereas Figure 4 mentions the use of curviameter data at a rehabilitation-level project.

### 3.7. Economic importance of using high-speed data survey systems at Project or Rehabilitation level

Currently, most pavement characteristic parameters are surveyed by systems providing high-resolution data and extensive sampling (IRI, cracks, rutting, skid resistance, etc.). Nevertheless, pavement deflections are often assessed by data which are widely separated from one another. In this case, the problem is that some important information of the pavement real condition is not registered. This implies that optimal maintenance and rehabilitation solutions are not always adopted.

An example is included in Figure 5. Pavement deflections of a road section (10 km length) are represented (black line includes 1 data every 5m, blue line includes 1 data every 500m and red line includes 1 data every 1000m). Subsection 1 contains, in general, low deflection values (see black line), but if red or blue lines are considered, deflection values of this subsection 1 are significantly higher than the real values. On the other
hand, subsections 2 and 3 contain very high deflection values. Nevertheless, if red or blue lines are considered, these high values are not registered. Therefore, if the maintenance and rehabilitation solutions of this road section had been adopted using the red or the blue lines (1 data every 500m or 1000m) the solutions adopted for the subsection 1 would have been more than enough (less rehabilitation and maintenance costs are needed) and the solutions adopted for the subsections 2 and 3 would have been insufficient (more rehabilitation and maintenance costs are needed).

Figure 4. Use of curviameter data at a project level

In this regard, in order to prevent problems as those described in Figure 5, current Road Spanish Standard 6.3-IC (Spanish Ministry of Development 2003) establishes that “the maximum distance between two deflection consecutive measures cannot be higher than 20m”.

Figure 5. Pavement deflections of a road section (1 data every 5m, every 500m and every 1000m)

4. CONSTRUCTION PROCESS

Both at the last stage of a road construction process and at the final acceptance (relationship between construction and maintenance), the bearing capacity is mainly determined from the measuring of deflections. Therefore, what is described in section 3 about maintenance management is also applicable to these stages. On the other hand, a lower layers acceptance control (foundation and subgrade) is usually performed with devices that directly determine the dynamic modulus of materials. The principle is very similar, since said modulus is obtained from the deflections recorded when applying a certain load, even though these devices are specifically developed to work during the construction process.

4.1. Accepting Devices For Infrastructure Subgrades

All devices included in Table 1 can also be used during subgrade construction in order to evaluate its bearing capacity. Nevertheless, taking into account their mechanical characteristics and sensors, it would be better to use other measuring systems specially indicated for embankments’ and subgrades’ construction. There
are several devices aimed at the acceptance control of granular or treated materials (Table 2 summarizes the main existing ones in Europe).

Table 2. Accepting Devices for Infrastructure Subgrades

<table>
<thead>
<tr>
<th>MEASURING SYSTEM</th>
<th>TEST STANDARD OR DOCUMENT</th>
<th>MEASURING SPEED</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static load plate</td>
<td>NLT-357 (1998)</td>
<td>Punctual device (4-5/day)</td>
</tr>
<tr>
<td>Dynamic load plate (ϕ 300mm)</td>
<td>UNE 103807-2 (2006)</td>
<td>Punctual device (30/h)</td>
</tr>
<tr>
<td>Dynamic load plate (ϕ 600mm)</td>
<td>UNE 103807-1 (2005)</td>
<td>Punctual device (30/h)</td>
</tr>
<tr>
<td>Portancemetre</td>
<td>“Guide technique: Portance des plates-formes” (2008)</td>
<td>3.5km/k</td>
</tr>
</tbody>
</table>

Regarding these four devices, the first one refers to the static load plate test, a traditional system included in the Standards. Static load tests require auxiliary means, involve interruptions and delays which affect the works’ pace and their performance. The number of tests per day is very reduced (4-5 tests/day), which leads to an insufficient knowledge of the subgrade since very few tests are performed. Moreover, many recent experiences with traditional load plate tests make it clear that there are some difficulties when assessing whether the compacting obtained with that test is the right one or not. As CEDEX (Spanish Center for the Studies and Experimentation of Public Works) published in 2009 (Santiago 2009), there are many tests that prove how the static load plate shows high modulus values in many sections with poor compacting.

In turn, the dynamic load plate test (ϕ 300mm) is limited by the system’s measuring range (lower than 75MPa), so that it is common for results of relevant materials that are well compacted to be out of said measuring range (Jansen 2013). Thus, out of the four devices included in Table 2, this one study focuses on dynamic load plate tests (ϕ 600mm) and the portancemetre.

The use of high performance equipment able to yield a high volume of data in a reduced period of time – such as the dynamic load plate or the portancemetre – makes it possible to divide them into homogeneous sections without need of additional tests.

As for the distance between test points, CEDEX recommends in Spain the sub-ballast measure with dynamic load plate equipment (ϕ 600mm) using a sampling rate and report equal to 20 m.

4.2. Dynamic Load Plate (600mm Diameter)

The dynamic load plate [described in Spanish UNE 103807-1 (AENOR 2005) and French NF P 117-2 (AFNOR 2004) standards] is a pulse generator, which applies a dynamic load on the tested layer, equivalent to a 13t axis travelling at a speed of 60Km/h. The system is installed on a pick-up vehicle, while the loading system and sensors are protected from the dust and vibration produced during construction works. The compressive modulus under a dynamic load (Evd) is calculated from the Boussinesq relation (1885). The deflection of the soil and the force of the impact are measured through sensors installed on the plate.

4.3. Portancemetre

The portancemetre device (described in the technical guide, LCPC 2008) is composed of a wheel with an inner eccentric load that performs a circular movement perpendicular to the surface under study and contrary to the vehicle’s direction. This compaction device also provides, in a very simple manner, information about compaction level that is being achieved during the construction. The device is moved at 3.5Km/h, which enables to perform a test every 0.80m approximately. The system’s performance can reach approximately a measure of 20Km in a single workday.

4.4. Adjustment Factors

Both devices under study (dynamic load plate -600mm diameter- and portancemetre) provide the vertical modulus result of the tested materials under a certain dynamic load following the corresponding procedure. Opposite to what happens with deflections, the majority of current standards that refer to the minimum required values for foundation and subgrade compressibility moduli do not establish adjustment coefficients based on the existing environmental factors. The materials that form the subgrades are hardly affected by temperature variations (Chandra et al. 1988 and Wolfe et al. 1993), so the fact that no coefficient is applied here seems
logical. On the contrary, these materials are indeed sensitive to moisture changes. Therefore, an adjustment factor would be applicable to the moduli obtained according to the moisture at the time of the measuring.

Figures 6 and 7. Dynamic load plate and portancemetre

4.5. Statistical Analysis Of The Obtained Values

A series of homogeneous areas upon which to perform a data statistical analysis can be defined (since the sample population is high), and fix representative parameters for each area. This would allow the analysis of the entire subgrade homogeneity, which ultimately leads to the compliance of the current standard recommendation with regard to the search of its structural capacity uniformity.

In this regard, the “Guide technique: Portance des plates-formes. Mesure du module en continu par le Portancémètre” (LCPC 2008) the following platform classification according to their homogeneity: Homogeneous (C <15%), Half homogeneous (C is between 15% and 20%) and Not homogeneous (C >20%), where C is the quotient between the deviation and the mean from the moduli recorded by the portancemetre.

4.6. Economic importance of using high-speed data survey systems at Construction level

It is very important to carry out earthworks and subgrades correctly (to this regard, it must be remarked that their contribution to bearing capacity of a linear work is greater than 50%, as shown in the existing literature) in order to prevent infrastructure premature failure, which might cause high rehabilitation costs.

An example of this is the economic impact that a premature problem might have on compacted materials of a railway platform as opposed to the cost for suitable control during construction. This example refers to a high-speed railway, although the conclusions can be extrapolated to any road or linear work. Broadly speaking, the construction average cost for 1 km of high-speed railway might be between 5 million and 12 million euros, according to the complexity of alignment, the amount of necessary structures, etc. Out of this cost, around 2.5 million euros approximately might be related to earthworks; therefore, if the railway platform quality has not been controlled thoroughly (especially structural behavior homogeneity), the necessary investments on rehabilitation might reach 20% to 40% of that amount, meaning an amount between 0.5 and 1 million euros per kilometer. The cost of being provided with the necessary handover control devices in order to ensure the railway platform quality might be around 0.5% of the said amount, making the best out of the high performance of control systems for various sections under execution.

Thus, having high-performance devices that enable ensuring the quality of railway platform might imply just 0.5% out of the economic cost for rehabilitation in case of premature failure, without considering any additional costs such as the cost of users as a result of service interruption or because of the need to repair infrastructures that have just started operating.

5. CONCLUSIONS

The survey application horizon of the parameters that determine pavement structure bearing capacity comprises from the construction process to the maintenance management, and it is also used as a connection point between construction and maintenance.

Regarding maintenance management, the entire deflection bowl is the pavement’s main parameter (together with additional information, such as traffic, and pavement and subgrade structure) that enables to determine the pavement structural capacity, so as to estimate its service life and design rehabilitation actions.
The features of the curviameter device make it the most suitable system to measure deflection in a large road network with respect to a System of Pavement Management.

On the other hand, the lower layers accepting control (foundation and subgrade) is usually performed with devices that directly determine the dynamic modulus of materials. These devices are specifically conceived to work during the construction process. Due to their special features, dynamic load plate (600mm diameter) and portancemetre appear to be the most suitable systems for this task.

The usage of the adequate devices to evaluate both pavements' and subgrades' structure condition has a very important economic impact in the future infrastructure's maintenance.

REFERENCES

Gorski, M. (2005): Détermination de modules de couches recyclées a froid. XX Congrès belge de la Route, Belgium.
**PAPER TITLE**  
(90 Characters Max)  
THE UNFINISHED POLICY ON ROAD USER CHARGES AND ROAD PRESERVATION UNIT: INDONESIA’S HOMEWORK TO IMPLEMENT ROAD USER CHARGES

<table>
<thead>
<tr>
<th>TRACK</th>
</tr>
</thead>
<tbody>
<tr>
<td>AUTHOR</td>
</tr>
<tr>
<td>(Capitalize Family Name)</td>
</tr>
<tr>
<td>Max ANTAMENG Ph.D.</td>
</tr>
</tbody>
</table>

| E-MAIL |
| (For correspondence) |
| cenrmia@yahoo.com |

| CO-AUTHORS (S) |
| (Capitalize Family Name) |
| DR. Ir. Slamet Muljono M.Sc | Deputy Director of Implementation II -DG Highways | Directorate General of Highways-Indonesia | Indonesia |

**ABSTRACT:**

The Indonesian government’s commitment to road transport, especially in terms of reducing overall road transportation costs by appropriate road preservation funding has been conveyed through the Law of Traffic and Transportation no. 22/2009. The process to persuade the public of the importance of the Road Preservation function and preservation funding commenced in 2000 and the legislative branch of Government approved this as part of the Law No. 22/2009.

The Regulation is not complete yet, several aspects need to be developed and explain detail such as: (1) sources of funding; (2) the division of funding between different road status; (3) the mechanism for collecting and distributing the road preservation funds and (4) setting up the road Preservation Unit or Board as an organization taking care of road preservation related issues.

The imbalance between traffic growth with the development of the road sector is very significant, so there needs to be a satisfactory way of anticipating the user demand with the supply of road transportation capacity for future as well as existing road users. Attention is needed to be focused on maintaining the condition of current roads in addition to increasing capacity, improving alignments and connectivity. This paper will discuss the funding model and the governance anticipated via a Road Preservation Board, with a review of the substance, background and details to be accommodated in new Government Regulations.

Keywords: Law No.22/2009, Unit Preservation Fund, National Road, Provincial Road, District/urban Road, Road User Charge
Chapter 1
Introduction

Indonesia currently finance the road through the government budget, while the government budget comes from general taxes including road user tax. Existing funds should be in the other departments that also would require funding. Basic decision to allocate funds through the government budget is political considerations rather than economic considerations.

Payments through the government budget, it's not a favorable options. The weakness of the traditional budget mechanisms have been put forward by Faiz and Harral (1988), Heggie (1994,1995, 1998), Boursquet and Fayard (1998), I Smith (1988), World Bank (1994) who argue that the road sector financing through mechanisms budget can no longer meet the needs of road. Road transport in industrialized countries, developing and other developed countries rose an average of 1.5 up to 2% x of GDP growth. This is very high compared to the growth of government revenue through taxes, making it difficult for the government to be able to meet all the financial needs such as: maintenance, improvement, modernization of road network, especially those for the regional (provincial+kabupaten+urban) road.

Empirical evidence in Indonesia, especially for Provincial and Kabupaten Road explains that the ability of the regional (provincial + District/kabupaten)government in providing the necessary funds have been insufficient from year to year, which in turn will affect the service quality of the existing infrastructure. If this continues, then the construction of road networks we have implemented together would be in vain, because eventually the road will be damaged. This of course will have serious affect the level of ability to compete in the global arena.

After 10 years effort to socialize the Road Fund/road preservation fund (introduced in Indonesia since year 2000), the government success to launch Preservation Fund unit in Indonesia through the enactment of Act No. 22/2009. The detail of Preservation Fund will be explain to this paper.

2 The role of road infrastructure in economic growth

There have been many study reviewed the role of the sector in economic growth. Some of them are Leff (1984) who said that the road infrastructure lowers transport costs and this has implications for increasing the efficiency of the institution. Similarly, OECD (1995) explains that road transport will foster economic development and social existence. In addition, road transport can also reduce the growing inequality between regions in a country or between countries. Canning and Fay (1993) adds that the rate of return from transport can exceed 200% in poor countries, newly industrialized countries and about 50% for countries emerging.

In the study of the relationship between transport investment with its impact on income distribution in isolated places (Louis Berger 1979), it could be concluded as follows (see diagram 1) the following:
1 There are three direct and immediate impact of the opening up of the isolated effects of (i) the creation of employment opportunities; (ii) accessibility, and (iii) the environment.
2 Decrease level of poverty on the one hand and improved quality of life on the other.
3 Increase in the potential socio-economic

Burky et al (1999) also argued that the existence of roads sector stimulates large tax revenues through the implementation of road user rates (Road User Charges) as Fuel Tax, Toll, Rates of motor vehicle records, driver's license, vehicle registration. Road sector is also a huge business, its contribution to total GDP is 6%.

Diagram 1 Impact of Investment in Transportation to the revenue distribution, sources: louis Berger
Chapter 2

Road Sector Financing

In this traditional budget system, then each ministry will compete to obtain funding. Although the theory is that the funds are given based on the expenditures which have economic and social returns are high, but the implementation

is more political. According to Heggie’s 1994 and Zietlow 1999, studies conducted in different parts of America, sub-Saharan Africa, found that neglect of maintenance of $1 will result in road users have to pay $0.3 because of damage to his vehicle along the damage road as well at the same time the government loss of $0.3 because of early rehabilitation and reconstruction of roads. Experience also shows that carrying out maintenance on time, will result in savings of $0.14 (ADT 300) up to $0.44 (10,000 ADT).

Financing through the government may be in various forms such as through general taxes or Earmarking tax. Examples of government funding from general taxation, can be seen in the general fund and special funds given to the region. General fund by fund basis formula, area, natural resources (mining, forestry and fisheries) and industry as well as incentives increase revenue. The use general funds left entirely to the appropriate regional priorities and needs. While dedicated to the Special Fund in accordance with the priorities and needs for specific areas such as road improvements, operation and maintenance as well as development of the region.

Funding from earmark tax been implemented by many countries in sub-Saharan Africa, Latin American countries, the United States and Japan, is to be re-used to the road sector. In addition to earmarking, it is known how much revenue the way that can be used for roads. It’s just that this earmarking system, which by Heggie (1998) is mentioned as the first-generation road fund. Earmarking system is only successful in the United States, which until now still use the earmarking system.

### Table 1: Administrative characteristics of different road user charging mechanisms, source: Heggie, 1994

<table>
<thead>
<tr>
<th>Charging instrument</th>
<th>Potential role</th>
<th>Related to road use</th>
<th>Separable from general taxes</th>
<th>Easily recognizable</th>
<th>Collection cost (%)</th>
<th>Administrative Characteristics</th>
<th>Ease of collecting by contract</th>
<th>suitability(a)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tolls</td>
<td>User fee</td>
<td>yes</td>
<td>yes</td>
<td>excellent</td>
<td>10-20</td>
<td>moderate</td>
<td>simple</td>
<td>moderate</td>
</tr>
<tr>
<td>Vehicle access fee</td>
<td>Vehicle access fee</td>
<td>no</td>
<td>yes</td>
<td>good</td>
<td>10-12</td>
<td>high</td>
<td>moderate</td>
<td>high</td>
</tr>
<tr>
<td>Fuel levy</td>
<td>not directly</td>
<td>yes</td>
<td>good</td>
<td>unknown</td>
<td>5</td>
<td>high</td>
<td>simple</td>
<td>high</td>
</tr>
<tr>
<td>Weight-distance fee</td>
<td>partly</td>
<td>yes</td>
<td>good</td>
<td>negligible</td>
<td>10</td>
<td>high</td>
<td>simple</td>
<td>high</td>
</tr>
<tr>
<td>International transit fee</td>
<td>partly</td>
<td>yes</td>
<td>good</td>
<td>unknown</td>
<td>over 50</td>
<td>high</td>
<td>simple</td>
<td>high</td>
</tr>
<tr>
<td>Parking charges</td>
<td>partly</td>
<td>yes</td>
<td>good</td>
<td>unknown</td>
<td>10-15</td>
<td>unknown</td>
<td>simple</td>
<td>moderate</td>
</tr>
<tr>
<td>Area License</td>
<td>partly</td>
<td>yes</td>
<td>moderate</td>
<td>unknown</td>
<td>10-15</td>
<td>unknown</td>
<td>simple</td>
<td>low</td>
</tr>
<tr>
<td>Electronic road pricing</td>
<td>partly</td>
<td>yes</td>
<td>moderate</td>
<td>unknown</td>
<td>10</td>
<td>unknown</td>
<td>simple</td>
<td>moderate</td>
</tr>
<tr>
<td>Cordon charge</td>
<td>partly</td>
<td>yes</td>
<td>moderate</td>
<td>unknown</td>
<td>10-15</td>
<td>unknown</td>
<td>simple</td>
<td>moderate</td>
</tr>
<tr>
<td>Congestion charge</td>
<td>partly</td>
<td>yes</td>
<td>moderate</td>
<td>unknown</td>
<td>10</td>
<td>unknown</td>
<td>simple</td>
<td>moderate</td>
</tr>
<tr>
<td>User or congestion charge</td>
<td>can be</td>
<td>yes</td>
<td>good</td>
<td>unknown</td>
<td>10</td>
<td>unknown</td>
<td>simple</td>
<td>low</td>
</tr>
</tbody>
</table>

Chapter 3

The Road Preservation Fund and Unit

As mentioned earlier that the issue of financing is part of the management problem, for it required an approach to handle the problem in an integrated and holistic (Dunlop, 1996, Talvitie, 1996, Heggie 1998). Financing problems should be approached not only by increasing the funds but also should improve efficiency through improved management system now (ESCAP 1996). Increased efficiency will lead to decreased costs, and decreased costs will lead to decreasing the total cost of the program and the way of handling this would certainly lead to more number of roads that can be handled.

The Government finally setup Road Preservation Unit on Act No. 22/2009 on Traffic and Road Transportation No. 22 of 2009 in Article 29, s / d 32 were declared and agreed on the use of road maintenance levy collected from road users. However, there are several aspect to be detail such as, among others: (1) Source of Funds of Road Preservation; (2) The procedure for the collection of funds; (3) Procedures for the distribution of funds; (4) the determination of the designated bank to store and distribute funds; (5) The organization model of Road Preservation Units, whether by province or between provinces combined with the organizing national road preservation
financing; members in the preservation unit both elements and stakeholders involvement; The division of labor, and so forth, which should be answered or determined based on Government Regulation.

Based on international empirical of road funds, there are 4 aspect have to be include on the setting up of road preservation fund or road fund as follows:

2.1. Clear Responsibility

Defining who is responsible for what, an essential element that must-have if you want to commercialize the road. Commercialization requires clear accountability on the road network. Clear accountability between the central, provincial, district, city, and even districts had to be outlined, especially in facing the implementation of Law no. 22/99 on decentralization.

World Bank (1994) suggests that in a survey of 44 developing countries to implement decentralization found the handling of the fact that the countries are decentralizing the management of the road sector, the ratio of paved road to total road network is greater than the countries that are not decentralized. In addition, the ratio is in good condition to the total length of the road network more than the countries that are not in decentralization.

Humplick Frannie and Moini - Araghi (1996) concluded that decentralization, will directly impact the cost ratio can be saved. They also get the fact that when the road sector was submitted to the local government, in the beginning investment and maintenance costs will be more expensive than the sentralilasi, this is more due to economies of scale, but it will eventually be closed at the time of an increase in namely during the organization's efficiency is more user-oriented way which is actually a public road.

2.2. Creating ownership

Building a partnership with the parties who have a great interest on the road users. Involving road users as one of the stakeholders of road infrastructure will lead to the increase of "willingness to pay". Road users should be involved in the management of the road. Road users, businesses, associations of farmers, transport companies and other people who depend on good road conditions are the ones who should be involved in the management of the road. Underfunded road handling is becoming a universal problem now becomes the burden of the stakeholders, and if the road users would believe the proper functioning of the management, they will want to apply / assist in achieving an optimal state of sustainable funding.

Ways to involve users in order to foster the willingness to pay was to enroll them in a workshop discussing the future of financing and road handling organization in Indonesia in general and the region in particular. Involving the group of road users in the management of a novelty and private sector participation can grow any business climate, efficiency in execution of work. Participation of road users are also opening our nature to be more transparent and more "clean" in the implementation of management tasks.

Groups representing users of the road and have a great interest is Gapensi, Jasamarga, oil companies in Riau, Chamber of Commerce, academia, local government, land transport companies, the Ministry of Agriculture, even if it needs to be established association Indonesian road users.

"Road Board" is the popular name of the manager of this road management, road board has been operating in Australia (The states of New South Wales, Queensland and Victoria), Finland, Ghana, India, Japan, Latvia, New Zealand, South Africa, Sweden, Britain, Yemen and Zambia. Some of these countries have a board to manage the road network of major roads, such as FinnRA, Ghana, New Zealand and South Africa. Road board in Japan, known as road council charged with providing advice to the Ministry of Construction of Japan as well transfund New Zealand.

The things that are important and need to be considered in establishing road board are: (i) the legal procedures to make the board, (ii) membership and procedures in selecting board members and appoint the chairman of the board; (iii) The role of the community; (iv) The amount and composition of the secretariat that manages the
daily business in the board ; ( v ) The function of the Board as well as the TOR including the relationship between
the Department of the managing board and the last path ( vi ) efforts are needed in order to board the transparent and
free from corruption .

2.3. Guarantee of sustainable funds

At this stage it is necessary to model that can promote economic efficiency and generate funds that is too to operate
and maintain the road network in the spectrum of a long time . It required the instruments of the charge obtained
from road tax and the absolute requirement for it is : ( i ) tax and the rates should be easily identifiable ; ( ii ) relates
to the use of the road ; ( iii ) easily separated from indirect taxes or tariffs other services ; ( iv ) is in administration .
Following administration presented the characteristics of the applicable usage rates in some countries ( see Annex 5 ).
The main instrument to impose tariffs on road users are : vehicle taxes , tariffs or taxes on fuel , international
transit and toll rates . Only a few countries that use the extra tariff for heavy vehicles . It is important to note in
determining the use of the tax is to tax / fee rates should be the same as that used when the use of the road network .
For this there are two types of costs that need to be considered : ( i ) The cost of damage to the road surface due to
the passage of vehicles ( such as operation and maintenance costs ) and ( ii ) the cost of traffic congestion is a
negative impact on society . Financing road handling is meant to include the entire program such as maintenance ,
construction of new roads and rehabilitation . Some countries such as Guatamela , Malawi , Yemen including
Indonesia when present ) to finance the construction of a new road through the traditional way ie through the
development budget . While others , such as Georgia , Hungary , Japan , Korea , Latvia , New Zealand , Romania ,
Russia and South Africa to finance the construction of new roads through Rates to road users . For road maintenance ,
it is recommended financed through the use of tariffs which include : Rates of motor vehicles , the transfer tax ,
overloading fines and tariffs on fuel . Especially for the rehabilitation of roads may also use the road tariff .

Rates / charges are no longer in the transfer to the government budget but transferred to the special account for the
road or the road fund . Many variations of the model of the road fund . The purpose of the road was clear for setting
up a fund to help finance expenditures of the way , sort of the revenue of the government budget and transfer
directly to the account of the road . Some road funds are only used to finance the national road or main road ( South
Africa ) , again only a few states , provinces and the regions such as Argentina , USA , Russia and Latvia . In other
countries, the road fund to finance urban roads only . The vast majority of road funds to finance the entire road
network in the country concerned .

2.4. implementation of a sound business system

To achieve this , there are some things that need to be followed up as : ( i ) the existence of a clear mission of the
organization ; ( ii ) separating the strategy planning and management with the implementation of road works (which
may include the implementation of the work through contract ) ; (iii)The number of staff who would qualify to be
adequate ; (iv) a good management structure ; (v) appropriate management information systems ; ( vi ) commercial
accounting system ; ( vii ) autonomy broad enough to be able to manage the road network efficiently

Chapter 4
Conclusion

Preparation of detailed rules on the Road Preservation Unit including the technical aspects of the implementation
that has been mentioned above , combined with a network system Traffic and Road Transport .

Procedures for the collection and distribution of funds from the road preservation and to the preservation fund
account . Which stated that the proposed collection of funds is as follows:
In relation to the use of funds, national sharing formula between the areas is also a substance that clearly elaborated by the Draft Regulation.

1. Scope of financing, according to Law no. 22 of 2009 is a road preservation. Thus the construction of new roads will be the responsibility of the government. The choice is between a pilot project limited to routine maintenance program, or routine maintenance and periodic maintenance, periodic or simply alone. Alternatively a pilot project carried out for all road preservation, but is confined to the district / city roads without provincial roads.

2. Operation of the unit is done with the preservation of the first pilot project. In the pilot phase of this project, will be selected provinces and districts which represent the western, central and eastern, of course, followers of this pilot project must have minimum standards or minimum requirements that must be followed. One important requirement is a willingness on the part of local governments to finance road maintenance in an optimal area, because there are still a lot of evidence to suggest that the maintenance of the road is not a main priority area of the Government even though they are aware of the importance of the conditioned steady path to economic growth and regional territory.

3. Procedures for surveillance and monitoring of the implementation of road preservation units, procurement procedures including the amount of the contract package is also part of the focus for the preservation unit. Road preservation projects are the future projects with a minimum length of about 100 km, which allows the output of the base contract also implemented.

4. Increased efficiency can be implemented through the commercialization of the road and began imposing a fee for services basis or user pay principles. Road users who use the road, should finance the construction and or maintenance. Commercialization delivering meaningful way as one of the products to be purchased by the speaker on the market for the management should be as business in general.
References

Antameng Max, 1997, Alternative ways for financing District road maintenance in Indonesia, paper presented to 10th environment seminar in Lima Peru, April

Antameng Max, 1998, Financing for road maintenance in Indonesia, paper presented to 2nd Indonesian Student Scientific meeting Paderborn-Germany, September

Antameng Max, 1998a, A National Policy framework for financing district road maintenance in Indonesia, Ph.D. theses, University of Leeds, England

Antameng Max and Dr. Patana Rantetoding, 1999, Reformasi manajemen dan pembiayaan pemeliharaan jalan dalam menyongsong era keterbukaan

Bousquet Frank, Fayard Main, 1997, Analysis of the interface between road financing and road management observation of current trends in Europe, PTRC: 24th European Transport Forum

ESCAP/World Bank, 1996, ESCAPIWorId Bank seminar on Management and Financing of road maintenance 17 to 20 September 1996

Faiz and Harral, 1988, Road deterioration in Developing countries, causes and remedies, A world Bank policy study, The World Bank Washington D.C.


Rantetoding P, 1999, Suatu agenda reformasi dalam pengelolaan dan pembiayaan pemeliharaan jalan, paper presented in Seminar pemeliharaan jalan, Jakarta, Dijen Bina Marga

Rupert Pennant-Rae and Ian Heggie, 1995, Finance and Development (Dec. 95), Washington-USA

Roth Gabriel, 1996, Roads in a Market economy, Brookfeld, United States of America


Ditjen Bina Marga, 1999, Memori tugas akhir Menteri PU, Jakarta, Indonesia

Heggie Ian G Dr and Piers Fickers, 1998, Commercial Management and Financing of roads, the World Bank, Washington, DC.

Hirotugu Doi (JICA expert in Bina Marga), 1999, Road Management System in Japan, Jakarta


Leff, 1984, Externalities, information costs, and social benefit costs analysis for economic development and cultural change, vol. 32, January, hal. 255-276

OECD, 1995, Road maintenance management systems in Developing countries, Paris

Schiessler Andreas and Alberto Bull, 1993, Roads: A new approach for road network management and conservation, United Nations Economic commission for Latin America and the Caribbean, June, Santiago-Chile


Zietlow Gunther Dr, 1999, Reform of financing and management of road maintenance, International Road Federation (IRF) dan Deutsche Gesellschaft fur Technische Zusammenarbeit (GTZ) GmbH.
Ordered Logit Model for Severity analysis of the accident on Thailand Rural Road Network

<table>
<thead>
<tr>
<th>AUTHOR (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>JERMPRAPAI, Khajonsak</td>
<td>Practical Engineer</td>
<td>Department of Rural Road</td>
<td>Thailand</td>
</tr>
</tbody>
</table>

E-MAIL (for correspondence) kjermprapai@drr.go.th

KEYWORDS:
Road Safety, Severity Model, Rural Road, Ordered Choice Model

ABSTRACT:
Road accident is a problem of the major concerns in Thailand. The fatality rate from road accident is 17.7 per 100,000 people, which is almost double of the average rate in Europe. Combining the damage from 100,000 road related injuries per year and other types of damage from the accident, the loss caused by road accidents is around 6,250 million US dollars per year. So the reduction of accident severity is one of Department of Rural Road’s (DRR) most important mission. To achieve this goal, more understanding on factors that affect the severity level is needed. With the introduction of new accident record collector system within DRR, the more in-depth analysis of accident severity is now possible. In this study, accident severity is being analyzed by the method of Ordered Logit model. 7 groups of risk factor which represent 4 main causes of road accident (driver, vehicle, road and environment) are included in the analysis. The proposed model shows that the degree of accident severity does not always directly proportional to the amount of traffic on the road. The result also implies that despite being the low to moderate volume road, the basic prevention for severe accident case could not be neglected. Furthermore, the proposed model also suggested that there are some biases that affected the accident records which need to be account for in the future.
Ordered Logit Model for Severity analysis of the accident on Thailand Rural Road Network

Khajonsak Jermprapai

1Department of Rural Roads, Bangkok, THAILAND
Email for correspondence: kjermprapai@drr.go.th

1 INTRODUCTION

Department of Rural Roads (DRR) is the public organization that responsible for construction and maintenance rural road network in Thailand. Rural roads of Thailand often serve as the collector/feeder for the highway network. They have the major role in the quality of life improvement in the rural and suburban area. The existence of this organization is due to the difference in characteristic from the high-volume arterial highway. The work of DRR is different from its counterpart which is responsible for major highway, the department of highway. According to the present, traffic volume of road in responsibility of DRR is around 1000 PCU/day though it could varied from as low as 100 PCU/day to around 15,000 PCU/day.

Road accident is a problem of major concerns in Thailand. The fatality rate from road accident is 17.7 per 100,000 people, which is almost double of the average rate in Europe. Combining the damage from 100,000 road related injuries per year and other types of damage from the accident, the monetary damage caused by road accidents is around 6,250 million US dollars per year. The chance of severe accident are not limit to only high volume road, in fact many of severe case are happen in low volume road of rural area. So the reduction of accident severity is one of Department of Rural Road’s (DRR) most important mission. To achieve this goal, more understanding on factors that affect the severity level is needed. With the introduction of new accident record collector system within DRR, the more in-depth analysis of accident severity is now possible. The goal of this study is to develop accident severity prediction model for rural road network from the improved accident reporting system.

2 LITERATURE REVIEW

Severity level of accident are usually categorize as ordinal variable. Ordinal variable is a type of categorical variable that have intrinsic. The example of other ordinal variable are opinion rating (bad – good), educational level, etc. The spacing between each order level is not always equal; for example, the spacing between college education and high school must be a lot larger than the spacing between junior high school and high school. Due to this fact, the ordinary least square regression could not be used. Past research usually use either Ordered probit model (Kockelman and Kweon (2012), Abdel-Aty (2003), Quddus et Al (2002)) or Ordered logit model (Al-Ghamdi(2012), O’Donnell and Connor (1996)).

In this study, ordered logit model has been used. The ordered logit model is a type of maximum likelihood based regression model method that could handle ordinal variable. The general form of ordered logit model is consisted of utility function and intercept. The general form of ordered logit model is shown in the equation below.

\[
y = \begin{cases} 
A; & \text{if } y' \leq u_1 \\
B; & \text{if } u_1 < y' \leq u_2 \\
C; & \text{if } u_2 < y' \leq u_3 \\
D; & \text{if } u_3 < y' \leq u_4 \\
\vdots & \\
\end{cases}
\]

(1)

Where; \( u_i \) = interception for each outcome
A, B, C, D,... = Ordinal variable of the event outcome
In the context of severity analysis, there are 2 groups of model structure. First is person level model. This type of model is focusing on the relation of socio-economic data of accident victim and other factor with the injury type each person sustained. The example of these types of model is Zambon and Hasselberg (2006). This type of model has the advantage of the more in-depth analysis of victim socio-economic factor that affect severity model. However, to establish this type of model, the characteristic data of each victim is needed. Most of the time, these data might not available due to the privacy reason. So the modeler might need to match the record by the mean other than surveying. The other type of model is event level model which considered the severity level of the accident. This type of model is focus on the physical aspect of the event such as geometric of the site and time of the event. Event level model require less victim identification data than person level model and more suitable for the highway organization. The examples of these types of model are (Kockelman and Kweon (2012), Abdel-Aty (2003), Al-ghamdi (2012), O'Donnell and Connor (1996), J Lee, F Mantering (2002), Quddus et Al (2002)).

The injury type and level of severity is another topic that worth discussion. FHWA use 6 injury types (K (Fatal), A (Disabling), B (Evident), C (Complaint), O (No injury) and U (Unknown)). The severity of the event is determined as the most severe injury type that any person in the event sustained. For example, if there is at least 1 fatality at the site of the accident, the accident will be labeled as the most severe K (Fatal) type accident disregarding how many people who have other injury type.

The severity level in Thailand is much simpler; there are only 4 injury types which are no injury, minor injury, severe injury and fatality. The definitions of each level are shown on Table 1.

Table 1 Thailand injury type classification

<table>
<thead>
<tr>
<th>Severity Level</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>PDO</td>
<td>Property damage only : no report of injury or fatality</td>
</tr>
<tr>
<td>Minor Injury</td>
<td>The victim have visible or complain about injury but do not admitted to hospital.</td>
</tr>
<tr>
<td>Severe Injury</td>
<td>The victim had been admitted and stay in hospital at least 24 hours.</td>
</tr>
<tr>
<td>Fatality</td>
<td>The victim is dead at the site of accident.</td>
</tr>
</tbody>
</table>

While there are the difference in detail of how to determine the injury level of victim, Thailand’s minor injury is the combination of US’s B and C injury type. Currently, there is no official guideline for the event level severity in Thailand.

Past severity model has variety of independent variable. However, these variables could categorize in to 2 groups. First socioeconomic variable, this type of variable reflect the demographic data of accident victim. This type of variable is usually found in person level model. This type of variable are generally extracted from either victim interview or census data. However, due to the privacy reason, this type of data is generally lacking in Thailand.

The second group of variable is geographic data of the accident site and the circumstance of the accident. These variables are the variable that hints about the cause and collision pattern of accident. Accident level model are generally consist of this type of variable Kockelman and Kweon (2012), Abdel-Aty (2003), Al-ghamdi(2012), O'Donnell and Connor (1996)). The examples of this type of variable are crash location, weather condition, light condition, time of the crash and, suspected violation.

3 METHODOLOGY

The goal of this study is to construct accident event level severity prediction model for Thai rural road network. The severity of the accident is defined as the highest injury that the victims sustain from the event. There are 4 injury types in this study as shown in Table 1. Ordered logit model has been selected for the analysis. The probability that the accident will have each level of severity could be calculated as followed (Equation (2)).
\[ y = \sum_{i}^{n} C_{i}x_{i} \]

Where \( C_{i} \) = Utility coefficient of each variable

\[ x_{i} \] = independent variables

\[ y = \begin{cases} 
PDO; & \text{if } y' \leq u_{1} \\
\text{Minor Injury}; & \text{if } u_{1} < y' \leq u_{2} \\
\text{Severe Injury}; & \text{if } u_{2} < y' \leq u_{3} \\
\text{Fatality}; & \text{if } u_{3} < y' \leq u_{5} 
\end{cases} \]

Where; \( u_{i} \) = intercept point for each outcome

\[ p_{i} = \frac{e^{\mu_{i}}}{\sum_{i}^{n} e^{\mu_{i}}} \]

Where; \( p_{i} \) = probability that outcome i will occur

4 DATA

This study use accident record data from Accident Record Management System (ARMS). The period of analysis is year 2009-2013. The report cards of ARMS contain the information about victim, number of injury, fatality, possible violation and surrounding circumstance of the accident event. The example of ARMS report card is shown in Figure 1.

![Figure 1 ARMS system accident report card](image)

There are 2143 records of accident after cleaning process. Severity of the accident is the dependent variable in this study. As state before, the severity of each accident defines as the highest injury type that the victim suffers. The severity distribution is shown on Table 2.
Table 2 Severity distribution

<table>
<thead>
<tr>
<th>Severity Level</th>
<th>Frequency</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>PDO</td>
<td>532</td>
<td>24.8%</td>
</tr>
<tr>
<td>Minor Injury</td>
<td>280</td>
<td>13.1%</td>
</tr>
<tr>
<td>Severe Injury</td>
<td>1148</td>
<td>53.6%</td>
</tr>
<tr>
<td>Fatality</td>
<td>183</td>
<td>8.5%</td>
</tr>
<tr>
<td>Total</td>
<td>2143</td>
<td>100.0%</td>
</tr>
</tbody>
</table>

From Table 2, we could see that severe injury has the highest percentage of sample share which is somewhat contradict the ordinance variable assumption. Perhaps this issue is the problem of bias report. Past researches (Alsop and Langley (2001), Yamamoto et AI (2008)) suggest that the accident of less severity (for example PDO and Minor injury) are tend to be under reported. The reason behind this problem is the fact that most of the accident data are from police organization. Per law, the police have to investigate all severe and fatality case. Because of this, high severity accidents are generally better documented than lower severity accident. Moreover, the person involved in less severe case are likely to escape the scene without report or be released without further investigation unless they cause property damage to the infrastructure. All of these reasons contribute to the problem of under-reporting on lower severity accident.

The other problem is the definition of severe injury and fatality. At the current state, the severe injury is defined by the need to be admitted as IPD (stay in the hospital for more than 24 hours). However, there is very large gap of actual severity in this definition. The accident victim could be admitted as a precaution in some case, while others may succumb to the wound in hospital. These 2 cases show the variety in the actual severity of the current definition of severe injury and also the reason that severe injury has the highest sample share.

The independent variables are consisted of the circumstance that has been record on the accident characteristic (see Figure 1 ARMS system accident report card) and route characteristic data. There are 7 groups of independent variables; location of the accident, weather condition, light condition, vehicle and pedestrian, crash pattern, suspected violation and route characteristic. The frequency distributions of independent variable are shown in Table 3. Please note the variables which do not have significance share are omitted from the table.

First group of independent variable is location of accident. These are the variables that give the context about what is the crash site look like. This variable is record by the authority that is at the site of accident. This type of variable is a one-to-one dummy variable which mean that each record could only have one type of location recorded. From Table 3, most of the accidents are happen on the straight section of the road follow by curve section and intersection. These locational characteristic are likely to have different effect on accident severity.

The second group of independent variable is the weather condition of the crash site. It is also one-to-one dummy variable. This group of variables represent the different driving condition that affect by the weather. The assumption is that the surrounding driving condition that changes due to the weather; for example, in the rainy condition pavement are tend have less friction while the visibility is less than desirable in the case of foggy weather.

Next is the light condition which is also one-to-one dummy variable. Past research (Jermprapai and Srinivasan (2014), Chang and Wang (2006)) suggest that the accident that happen outside of day light are tend to be more severe. In this study, there are 3 kind of light condition, day light, night – enough lighting and, night – not enough lighting.

Vehicle and Pedestrian is next. One unique characteristic of Asian highway is the number of motorcycles. Unlike US and European counterpart, motorcycles are used widely in Asian country. The safety problems of motorcycles are well known as they are tending to have worse safety equipment than personal...

481
car or truck. So the severity of the accident that involved motorcycle is usually resulting in fatality. Similar to the motorcycle, pedestrian is one of the most vulnerable parties on the road. However, the data indicate that the sample size of pedestrian accident is rather small so the significant of the effect is in doubt. Last but not least is the single vehicle accident that has been expected to have different in severity level from other type of vehicle.

Table 3 Descriptive Statistics of independent variable

<table>
<thead>
<tr>
<th>Variable</th>
<th>Count</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Location of Accident</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Straight</td>
<td>1057</td>
<td>49.30%</td>
</tr>
<tr>
<td>Curve</td>
<td>537</td>
<td>25.10%</td>
</tr>
<tr>
<td>Intersection</td>
<td>269</td>
<td>12.60%</td>
</tr>
<tr>
<td>Bridge</td>
<td>38</td>
<td>1.80%</td>
</tr>
<tr>
<td>Median Open</td>
<td>33</td>
<td>1.50%</td>
</tr>
<tr>
<td><strong>Weather Condition</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dry</td>
<td>1898</td>
<td>88.60%</td>
</tr>
<tr>
<td>Rain</td>
<td>84</td>
<td>3.90%</td>
</tr>
<tr>
<td>Foggy</td>
<td>26</td>
<td>1.20%</td>
</tr>
<tr>
<td><strong>Light Condition</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Daylight</td>
<td>1131</td>
<td>52.80%</td>
</tr>
<tr>
<td>Night - Enough Light</td>
<td>690</td>
<td>32.20%</td>
</tr>
<tr>
<td>Night - Not Enough Light</td>
<td>128</td>
<td>6.00%</td>
</tr>
<tr>
<td><strong>Vehicle and Pedestrian</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Single Vehicle Accident</td>
<td>120</td>
<td>5.60%</td>
</tr>
<tr>
<td>Motorcycle Involved</td>
<td>414</td>
<td>19.30%</td>
</tr>
<tr>
<td>Pedestrian Involved</td>
<td>69</td>
<td>3.20%</td>
</tr>
<tr>
<td><strong>Crash Pattern</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>At intersection</td>
<td>86</td>
<td>4.00%</td>
</tr>
<tr>
<td>Head On</td>
<td>52</td>
<td>2.40%</td>
</tr>
<tr>
<td>Back Collision</td>
<td>114</td>
<td>5.30%</td>
</tr>
<tr>
<td>Side Swipe</td>
<td>173</td>
<td>8.10%</td>
</tr>
<tr>
<td>Vehicle Overturning</td>
<td>142</td>
<td>6.60%</td>
</tr>
<tr>
<td>Roadside Hazard</td>
<td>70</td>
<td>3.30%</td>
</tr>
<tr>
<td><strong>Suspected Violation</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Illegal Speeding</td>
<td>712</td>
<td>33.20%</td>
</tr>
<tr>
<td>DUI/DWI</td>
<td>252</td>
<td>11.80%</td>
</tr>
<tr>
<td>Unsafe Cutting off</td>
<td>330</td>
<td>15.40%</td>
</tr>
<tr>
<td>Illegal Passing</td>
<td>86</td>
<td>4.00%</td>
</tr>
<tr>
<td>Dozed off</td>
<td>49</td>
<td>2.30%</td>
</tr>
<tr>
<td><strong>Route Characteristic</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length</td>
<td>13.53</td>
<td>18.34</td>
</tr>
<tr>
<td>PCU</td>
<td>5188</td>
<td>8370.22</td>
</tr>
</tbody>
</table>
Crash pattern is the next group of variable. It is the variable that reflects the collision type of the accident. Each type of crash patterns are expected to have different severity level. For example, the head on collision are expected to cause higher severity accident than side swipe. As some crash pattern such as crash at intersection are unique to the specific location type. With this reason, the problem of multicolinearity exist and must be check in the final model. Also there are many reports that do not record the exact crash pattern which might affect the analysis result.

Suspected violation is the variable that represent the opinion of accident reporter about the violation that contribute to the cause of accident. This type of variable is many-to-one dummy variables which mean that each record could only have more than one violation record. Illegal speeding has the highest sample share at 33.2%. The collision speed is a factor that should contribute to the severity of the accident. The other factors that have high sample share are unsafe cutting off and DU/DWI which should have increase accident severity level. One thing that should have been considered is that this variables group is based on reporter opinion. So the bias of reporter could sway the analysis.

Last but not least is the route characteristic which represent the overall characteristic of the route that accident happen. It is consisted of length and traffic volume of the route. While rural roads in Thailand are generally have traffic volume of 1000 PCU/day, the median PCU for the accident record is as high as 5188 which suggested that most accident occurred on the high volume road. The high standard deviation of the data shows the high variability of rural roads in Thailand.

5 EMPIRICAL RESULT

Table 4 shows the empirical result. T-test has been conducted to ensure that the variable has significance effect. The null hypothesis for T-testing is that the coefficient of tested variable could be 0. In other word, the confidence interval of the given confidence level must not contain the value of zero. At 95% confidence level, the T-value require to be higher than 1.96 to reject null hypothesis. Due to the concern about multicolinearity problem, Variance Inflation Factor (VIF) is included in Table 4. VIF is the value that reflects the severity of multicolinearity problem of regression model. Past research (Kock and Lynn, 2012) suggest that the multicolinearity would considered being high if VIF exceed 5.0. As the highest VIF of proposed model is 3.86, the multicolinearity of the proposed model is not severe and negligible.

All variables shown in Table 4 are significance at 95% confidence level. The positive coefficient of variable indicates of severity level increasing effect. The average effect of each variable could be compare by the model coefficient. To understand the effect of each coefficient correctly, the interpretation of coefficient effect need to be analyze at group level.

1. Effect of Location of Accident

The magnitude of model coefficient indicate that location of accident comparatively affect the severity level the most. While other type of variable has the coefficient value less than 1, this group of variables has the coefficient range of 2.0-2.5. Relatively, accident that happen on the straight and bridge section have relatively less severity level than other type of road section. Empirical also suggest that the accidents that happen at median open is the most severe (2.61). According to this, safety aspect of median open should be given special attention.

2. Effect of Weather Condition

Based on past research, the weather condition should have the effect on the severity level. However, weather seems to have no significance effect at 95% confidence level in this study. It is possible that the drivers are usually driven more careful in the case of bad weather and drive at less speed. So the increasing of safety awareness already offset the slippery and visibility hazard cause by bad weather.
3. Effect of Light Condition

The positive utility coefficient indicate that night time accidents that happen in the night are tend to be more severe than day time. This finding seems to be consistent with pass research. The adequacy of the light has the effect on severity too, as the accident that happen on the road section with not enough light (0.2931) are comparatively less severe than road section with not enough light (0.4524). The difference between night and day accidents suggests 2 possible explanations. First, the light provided from street light is not actually adequate as there is still have the severity level higher than day light. The improvement street light and night time navigation technology is still needed. The other explanation is that there is other unmeasurable factor affect the severity of night time accident. This factor might be the driver’s risk such as fatigue condition, eye vision and other. Anyway, the severity of night time accident should not be overlook and worthwhile to have further study.

Table 4 Empirical result

<table>
<thead>
<tr>
<th>Independent Variable</th>
<th>Coefficient</th>
<th>Standard Error</th>
<th>T-Value</th>
<th>VIF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crash Location</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Straight</td>
<td>2.2304</td>
<td>0.1801</td>
<td>12.3830</td>
<td>3.3678</td>
</tr>
<tr>
<td>Curve</td>
<td>2.2855</td>
<td>0.1891</td>
<td>12.0890</td>
<td>2.8698</td>
</tr>
<tr>
<td>Intersection</td>
<td>2.3963</td>
<td>0.2076</td>
<td>11.5440</td>
<td>2.1468</td>
</tr>
<tr>
<td>Bridge</td>
<td>2.2553</td>
<td>0.3691</td>
<td>6.1110</td>
<td>1.1793</td>
</tr>
<tr>
<td>Median Open</td>
<td>2.6110</td>
<td>0.3806</td>
<td>6.8600</td>
<td>1.1848</td>
</tr>
<tr>
<td>Light Condition</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Night - Enough Light</td>
<td>0.2931</td>
<td>0.0963</td>
<td>3.0430</td>
<td>1.1399</td>
</tr>
<tr>
<td>Night - Not Enough Light</td>
<td>0.4524</td>
<td>0.1914</td>
<td>2.3640</td>
<td>1.1300</td>
</tr>
<tr>
<td>Vehicle and Pedestrian</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Motorcycle</td>
<td>0.2636</td>
<td>0.1097</td>
<td>2.4020</td>
<td>1.0547</td>
</tr>
<tr>
<td>Crash Pattern</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Head On</td>
<td>0.7044</td>
<td>0.3058</td>
<td>2.3030</td>
<td>1.0438</td>
</tr>
<tr>
<td>Back Collision</td>
<td>0.3836</td>
<td>0.1746</td>
<td>2.1970</td>
<td>1.1025</td>
</tr>
<tr>
<td>Violation</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unsafe Cutting off</td>
<td>0.6376</td>
<td>0.1204</td>
<td>5.2940</td>
<td>1.0615</td>
</tr>
<tr>
<td>Route Characteristic</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Route Length</td>
<td>0.0092</td>
<td>0.0025</td>
<td>3.7290</td>
<td>1.0637</td>
</tr>
<tr>
<td>Traffic Volume (per 1000 PCU)</td>
<td>-0.0308</td>
<td>0.0055</td>
<td>-5.6240</td>
<td>1.1558</td>
</tr>
<tr>
<td>Intercept</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PDO -&gt; Minor Injury</td>
<td>1.0934</td>
<td>0.1768</td>
<td>6.1862</td>
<td></td>
</tr>
<tr>
<td>Minor -&gt; Severe Injury</td>
<td>1.8141</td>
<td>0.1799</td>
<td>10.0816</td>
<td></td>
</tr>
<tr>
<td>Severe Injury -&gt; Fatality</td>
<td>4.8902</td>
<td>0.1991</td>
<td>24.5573</td>
<td></td>
</tr>
<tr>
<td>Log-Likelihood at convergence</td>
<td>-2291.707</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Log-Likelihood at Null</td>
<td>-2477.906</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
4. Effect of Vehicle and Pedestrian

Pass research suggest that the accident that involved pedestrian are tend to have more severity level. In this study; however, the severity effect of pedestrian involved on accident is not significant. Perhaps, the sample size of pedestrian accident is too small as there are a mere 3.2% of pedestrian accidents in sample share. On the other hand, the involvement of motorcycle has the effect on severity level of accident on this sample share (0.2636). As the safety equipment of motorcycle are generally have lower standard than passenger vehicle. While passenger vehicle has safety frame, airbag and safety belt to reduce the collision damage, motorcycle only have safety. So the positive Severity increment property is understandable.

5. Effect of Crash Pattern

Head on collision and vehicle overturning have significant severity increasing effect. Head on collision accident (0.7044) is generally cause serious damage due to the direction of impact so this finding is consistence with intuitive. Similar with head on collision, the severity increasing effect of vehicle overturning (0.3836) is making sense due to the magnitude of impact that happens at the event. The crash at the intersection are also expected to have severity increasing property, however, the empirical effect suggest that it doesn’t significant effect. As stated before, the effect of crash pattern have some multicollinearity with the location accident. For example, crash at intersection can only happen at the intersection. Therefore, the inclusion of some variable in this group might not need.

6. Effect of Suspected Violation

The only variable in this group that has significant effect is unsafe cutting-off behavior (0.6376). It seems to be counter intuitive that over speeding do not have significant effect on severity level. Perhaps this is due to the fact that this variable might not reflect the collision speed of the vehicle as it’s not reflect the quantitative number of speeding. The quantitative data collection of speeding behavior might need for the analysis of the actual severity effect. Also it’s has to be noted that this type of variables is based on the reporter’s opinion. It’s possible that the analysis result could be sway by the bias and need further study to account for.

7. Effect of Route Characteristic

The length of route could act as the proxy variable for actual speeding behaviour. The longer route means that the drivers have longer speeding distance which leads to higher collision impact. Empirical result indicate that route length have the effect of increasing severity level of accident. The negative coefficient of PCU indicates that the more traffic volume the less likely that accident will be severe. There are several explanations to this finding. First, the routes with higher traffic volume are likely to have lower operating speed. The lower speed crashes tend to have less collision impact so the severity is undoubtedly lower. Especially, in the case of traffic congestion which the total number of accident will be higher but the most of the accident will have minor to low injury. The other explanation is the access time to the site of medical service. As the low volume roads are generally located in the remote rural area so the access time is much higher. The longer time that medical team need to reach the accident site could lead to the change of injury type as the condition of victim worsen by the time.

6 CONCLUSIONS

This study is about the developing of accident severity prediction of rural road network in Thailand. The data that use in this study is 2010-2013 accident data derived from DRR’s Accident Reporting Management System (ARMS). The empirical result suggest that some of crash location, light condition, vehicle & pedestrian, crash pattern, driver violation and, route characteristic have the effect on accident severity. Due to the magnitude of coefficient, empirical Accident location is the most important factor that affects the severity of accident. Some of accident location type needs to have more attention than others, especially median open and intersection which have higher utility coefficient than other.

The proposed model also suggested that night accident need to be given more attention. The coefficient of the model suggest that accident that happen on the night time is significantly more severe than
day time accident (0.2931,0.4524). The reason could be the either visibility condition or other unmeasurable factor. However, both are convincing that night time accident is a serious problem and deserved dedicated study. The empirical result also suggested that even though the technology of traffic light has been developed for so long, the condition is still not perfect compare to the daylight. So the continually development of traffic light technology is still required.

In term of special vehicle, motorcycle accident has been known for the long time about the high severity accident tendency. The proposed model confirms this hypothesis as well. So given that the utilization of motorcycle in Asian road network is always high, the improvement of motorcycle safety is needed. The improvement could be in the form of either regulation (safety helmet enforcement) or technology improvement. Also the dedicated study about motorcycle accident in term of both frequency and severity is needed.

There are several problems in current accident report found during this study. First problem is the under-reporting. This leads to the data bias to the high severity side. The fact that severe accidents have the highest sample share of all cast the doubt to the assumption that severity is ordinance variable a little bit. The better data collection or at least the study about under report factor can solve this problem. Another problem found in the study is the bias of report data. At the current state, the pattern of reporter’s bias is not been known. Some factor, especially, suspected violation variable affect by this problem. The calibration factor for reporter’s bias is needed in order to improve the accuracy of the analysis.

Finally, the effect of traffic volume in this study suggests that the accidents in lower volume route are not negligible. Even though the frequency of accident is much lower than high traffic road, the level of severity is higher. So the problem of accident in rural road with lower traffic volume require difference countermeasure than high traffic road. With this study, DRR have more understanding of the factor that influenced severity of accident which will lead to the better planning of countermeasure.

REFERENCE


Some concern for rational use of hydraulic graded iron and steel slag as reinforced base-course in Japan

**KEYWORDS:**
Iron and steel slag, Base-course, Uniaxial compressive strength, Hydraulic nature, Variability

**ABSTRACT:**
As is well known, iron and steel slag is an industrial by-product or waste in the process of making iron and steel. In Japan, three types of iron and steel slag products are currently used for base-course construction: crusher-run slag, graded slag and hydraulic graded slag. The hydraulic graded iron and steel slag literally exhibits a stronger hydraulic nature compared with other two slag products. By this, it has provided a more durable and longer life base-course. It is made by blending blast furnace slag and steel slag with or without additives. In this study, hydraulic graded iron and steel slag produced in three different manufacturers is investigated with a focus on its hydraulic nature. The uniaxial compression test was conducted on the specimens cured for different periods of time up to 730 days (two years). It is shown that there is a considerable variation among these three manufacturers in the magnitude and the increase rate of uniaxial compressive strength with time, indicating that constructed base-course may show a different performance depending on which hydraulic graded iron and steel slag is used. It is also indicative that, a difficulty could be encountered when one attempts to design the base-course thickness rationally.
Some concern for rational use of hydraulic graded iron and steel slag as reinforced base-course in Japan

Dr Nobuyuki Yoshida¹

¹Kobe University, Kobe, Japan
Email for correspondence: nyoshida@kobe-u.ac.jp

1 INTRODUCTION

Iron and steel slag is a by-product during the process for manufacturing iron and steel. In 2012, about 24,160 thousand tonnes of iron slag (blast-furnace slag) and 13,760 thousand tons of steel slag were produced in Japan; and as shown in Figure 1, their annual production for the past decade fluctuates between 21,600 and 25,400 thousand tons for the iron slag and between 12,100 and 15,200 thousand tons for the steel slag (Nippon Slag Association 2013). The iron and steel slag is used for various purposes. As shown in Figure 2, about 70% of the total use of iron slag goes to the cement production industry, and about 14% are used in road construction in 2012. On the other hand, regarding the use of steel slag, about 34% is used in civil engineering works and about 30% in road construction.

![Figure 1. Change in production of iron and steel slag in Japan.](image1)

![Figure 2. Uses of iron and steel slag in Japan in 2012.](image2)
Table 1. Iron and steel slag used for road construction in Japan (modified after JIS A 5015, 2013)

<table>
<thead>
<tr>
<th>Type</th>
<th>Crusher-run iron and steel slag</th>
<th>Graded iron and steel slag</th>
<th>Hydraulic graded iron and steel slag</th>
<th>Crusher-run steel slag</th>
<th>Single-graded steel slag</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Designations</td>
<td>CS-40, CS-30, CS-20</td>
<td>MS-25</td>
<td>HMS-25</td>
<td>CSS-30, CSS-20</td>
<td>SS-20, SS-5</td>
<td></td>
</tr>
<tr>
<td>Usage (course)</td>
<td>Subbase-course</td>
<td>Base-course</td>
<td>Base-course</td>
<td>Hot asphalt-stabilization</td>
<td>Hot asphalt mixtures</td>
<td></td>
</tr>
<tr>
<td>Coloration</td>
<td>No coloration</td>
<td>No coloration</td>
<td>No coloration</td>
<td>-</td>
<td>-</td>
<td>Only for blast-furnace slag</td>
</tr>
<tr>
<td>Immersion expansion ratio (%)</td>
<td>1.5 or smaller</td>
<td>1.5 or smaller</td>
<td>1.5 or smaller</td>
<td>2.0 or smaller</td>
<td>2.0 or smaller</td>
<td></td>
</tr>
<tr>
<td>Unit mass (kg/liter)</td>
<td>-</td>
<td>1.5 or larger</td>
<td>1.5 or larger</td>
<td>-</td>
<td>-</td>
<td>Only for steel slag</td>
</tr>
<tr>
<td>Uniaxial compressive strength (MPa)</td>
<td>-</td>
<td>1.2 or larger</td>
<td>1.2 or larger (13-day cured)</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Density in saturated surface-dry condition (g/cm³)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2.45 or greater</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Water absorption percentage (%)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>3.0 or smaller</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Abrasion (%)</td>
<td>-</td>
<td>-</td>
<td>50 or smaller</td>
<td>30 or smaller</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Aging</td>
<td>6 months or more</td>
<td>6 months or more</td>
<td>6 months or more</td>
<td>3 months or more</td>
<td>3 months or more</td>
<td>Applied to steel slag</td>
</tr>
</tbody>
</table>

In Japan, blast-furnace slag was first standardized as "Slag for road construction" in 1979, which designated its use for base- and subbase-course (Japan Standards Association 2013). When it was revised in 1992, steel slag was included and the Standard was renamed as "Iron and steel slag for road construction" in which applications to hot asphalt mixtures for surface- and binder-course were also adopted. The latest revision of the Standard was made in 2013, but the essence has not been changed. In the Standard, the iron and steel slag for road construction is classified into five types: crusher-run iron and steel slag, graded iron and steel slag, hydraulic graded iron and steel slag, crusher-run steel slag and single graded steel slag. The first three slags are literally made by blending iron and steel slags.

Among others, the hydraulic graded iron and steel slag consists of air-cooled blast-furnace slag, granulated blast-furnace slag, steel slag and some additives but their proportions depend on the manufacturers. The hydraulic graded iron and steel slag blended by some manufacturers may not contain granulated blast-furnace slag nor any additives. Table 1 summarizes some quality requirements extracted from the latest edition of the Standard (Japan Standards Association 2013). In the table, the items "coloration", "immersion expansion ratio" and "aging" appear to be unique to these slags. "Coloration" is to check whether or not sulfur in blast-furnace slag is sufficiently stabilized. The blast-furnace slag contains a small amount of sulfur in a form of calcium sulfide: upon contact with water, calcium sulfide is hydrolyzed and in the progress of successive reaction, polysulfide iron is produced of which solution exhibits a yellow color, often emitting a bad smell (commonly known as a "yellow water" problem). "Aging" is a treatment required for free lime in steel slag to be hydrated to suppress its expansion. A common aging treatment is simply to store slag in an open yard for the specified period of time; however, in present practice, warm water or vapour is also applied for accelerating the both reactions (Japan Slag Association 2004). "Coloration" and "immersion expansion ratio" are for confirming that sulfur and free lime contained in slag are sufficiently stabilized, respectively. As seen in Table 1, uniaxial compressive strength is designated only for the hydraulic graded iron and steel slag because the slag counts on a stronger hydraulic nature compared with the other two iron and steel slag base-course materials.

Among others, hydraulic graded iron and steel slag has demonstrated its favorable durability. In fact, as will be shown later, the equivalency conversion coefficient assigned for hydraulic graded iron and steel slag is the same as for cement-stabilized base-course material. Based on the follow-up investigation over five years on a trial asphalt pavement with a hydraulic graded iron and steel slag base-course, Yoshida et al. (2013) indicated that a better performance demonstrated by the pavement results from the long-lasting
Table 2. Equivalency conversion coefficients for pavement materials (modified after Pavement Design Manual, 2006)

<table>
<thead>
<tr>
<th>Location</th>
<th>Material</th>
<th>Quality specifications</th>
<th>a</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base-course</td>
<td>Hot asphalt mix</td>
<td>Refer to another sheet.</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>Bituminous stabilization</td>
<td>Hot-mixed: Marshall stability ≥ 3.43 kN</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cold-mixed: Marshall stability ≥ 2.45 kN</td>
<td>0.55</td>
</tr>
<tr>
<td></td>
<td>Cement stabilization</td>
<td>Uniaxial Compressive strength (7 days) ≥ 2.9 MPa</td>
<td>0.55</td>
</tr>
<tr>
<td></td>
<td>Lime stabilization</td>
<td>Uniaxial Compressive strength (10 days) ≥ 0.98 MPa</td>
<td>0.45</td>
</tr>
<tr>
<td></td>
<td>Graded crushed stone</td>
<td>Modified CBR ≥ 80</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Modified CBR ≥ 80</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td>Graded iron &amp; steel slag</td>
<td>Uniaxial Compressive strength (14 days) ≥ 1.2 MPa</td>
<td>0.55</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Modified CBR ≥ 80</td>
<td>0.35</td>
</tr>
<tr>
<td>Subbase-course</td>
<td>Crusher-run, iron &amp; steel slag, sand, etc.</td>
<td>Modified CBR ≥ 30</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>20 ≤ Modified CBR &lt; 30</td>
<td>0.20</td>
</tr>
<tr>
<td></td>
<td>Cement stabilization</td>
<td>Uniaxial Compressive strength (7 days) ≥ 0.98 MPa</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>Lime stabilization</td>
<td>Uniaxial Compressive strength (10 days) ≥ 0.7 MPa</td>
<td>0.25</td>
</tr>
</tbody>
</table>

development of hydraulic nature inherent in the base-course material. The author recalls, immediately after the occurrence of the Southern Hyogo prefecture earthquake in 1995, a road construction company conducted a reconnaissance investigation on roads damaged primarily by liquefaction and reported unofficially that the roads with a hydraulic graded iron and steel slag base-course subjected to less damage and that his investigation vehicle could go through on the roads without difficulty.

The mainstream design method of asphalt pavement in Japan is so-called $T_s$ method, an empirical one, which is based upon a subgrade CBR value and an equivalency conversion thickness ($T_s$) since 1967 (Japan Road Association 2006). The essential part is that: a pavement section is designed so as for $T_s'$ to be greater than the targeted $T_s$ estimated from the number of wheels and subgrade CBR. Here, $T_s'$ is computed as $T_s' = a_1T_1 + a_2T_2 + ... + a_nT_n$, where $T_s'$ is the equivalency conversion thickness of the designed pavement section, $a_i$ is an equivalency conversion coefficient for each pavement material used, and $T_i$ is the thickness of each layer. Table 2 summarizes the equivalency conversion coefficients assigned for pavement material. It is seen that 0.55 is assigned to the hydraulic graded iron and steel slag, which is greater than the one for lime stabilization and the same as for the cement stabilization.

In 1992, in order to facilitate the use of new pavement materials and structures and also a rational design of asphalt pavement, a mechanistic-empirical design method was first conceptually introduced in the Asphalt Pavement Manual (1992), which is based upon a multi-layered linear-elastic theory. Since then, more specific descriptions on this design method have been provided in 2001 and 2006 (Japan Road Association 2001, 2006). Nevertheless, it can be said that this design method is not yet adopted in practice. One of the crucial shortcomings is insufficient input data for pavement materials such as resilient modulus, Poisson's ratio, etc. Therefore, in order to use hydraulic graded iron and steel slag rationally, it is necessary to grasp its mechanical characteristics and to provide them as the input data for the structural design of asphalt pavement in a usable form, if possible.

The purpose of this experimental research program is to grasp the mechanical characteristics of hydraulic graded iron and steel slag obtained from three different manufacturers in Japan and to investigate whether or not the material can be used as a reinforced base-course rationally. In the followings, uniaxial compression tests on the specimens cured up to 730 days of the three different base-course materials are described focusing on the hydraulic nature of the base-course material.

2 HYDRAULIC GRADED IRON AND STEEL SLAG TESTED

The hydraulic graded iron and steel slag (occasionally called HMS hereafter) tested in this study was brought in from three different manufacturers in Japan in 2011. As stated earlier, the base-course material may consist of air-cooled blast-furnace slag alone, a blend of blast-furnace slag and steel slag, or other
Figure 3. Grain size distributions and compaction curves for three HMS tested. Combinations of iron and steel slags with or without additives. The ingredients and mixing proportions vary from one manufacturer to another, which are the manufacturers' secrets: no exception for the HMS tested here. Needless to say, all these base-course materials satisfy the requirements imposed by the Japanese industrial standard such as those in Table 1.

Figure 3(a) shows the grain size distributions for the three HMS. It is seen that all the materials are within the range designated in the Japanese Industrial Standard (2013). The densities of solids for each HMS in g/cm³ are as follows: 3.026 for HMS-1, 3.099 for HMS-2 and 3.154 for HMS-3. HMS-3 is the largest density of solids and HMS-1 is the smallest. The density of solids obtained for other HMS in the past ranged from 2.37 to 3.09 g/cm³; thus, the densities of these three HMS lie in the higher side. The difference in the density of solids may imply that HMS-3 contains more a heavier component such as steel slag.

Regarding the compaction characteristics, Figure 3(b) shows the compaction curves for the three HMS tested. Also shown in the figure are the data points of the maximum dry density and optimum water content obtained from the past experiments. The maximum dry density and optimum water content of each HMS are as follows: 2.130 g/cm³ and 8.3% for HMS-1, 2.085 g/cm³ and 13.6% for HMS-2, and 2.421 g/cm³ and 8.8% for HMS-3. HMS-2 exhibits the smallest maximum dry density and the largest optimum water content than the other two HMS. Compared with the past data, it seems that HMS-1 exhibits a lower maximum dry density for its optimum water content.

3 SPECIMEN PREPARATION AND UNIAXIAL COMPRESSION TEST

Specimens were prepared in the following manner. The water content of each HMS was first adjusted to its optimum water content with distilled water. Then, a prescribed amount of the material was placed into a mould in five stages, in each stage compaction being performed using a rammer with a mass of 4.5 kg, in such a way that the resulting dry density becomes 100% of its maximum dry density. Note here that grains not passing a 19.1mm sieve were excluded for the specimen preparation, considering the specimen size with a diameter of 100 mm and a height of 200 mm. Each specimen with its mould was double-wrapped up with polyethylene bags and a weight of 49N was placed on its top. Then, they were cured in a dark room with a humidity of about 60% and a temperature of about 20°C for prescribed periods of time: 0, 14, 24, 28, 90, 180, 365 and 730 days. The number of specimens prepared for each HMS was three for each curing time in principle.

Uniaxial compression test is a common test and was carried out with a loading rate of 1% strain per minute, following the Japanese industrial standard (2009). Note here that, as will be seen later, the reduced number of test results was obtained due to unexpected troubles in the test apparatus and the experimental facility.
Figure 5. Uniaxial compressive strength in the present and past studies for HMS-1 and HMS-2.

4 TEST RESULTS AND DISCUSSIONS

Only the uniaxial compressive strength is discussed here so that "some concern" in the paper title will be clarified. All the data of uniaxial compressive strength are plotted against curing time in Figure 4. Here, one may recall the quality requirement given in Table 1; that is, a uniaxial compressive strength should be 1.2 MPa or greater. From Figure 4, one may think that the three HMS used in this study did not satisfy the requirement. However, it should be mentioned that the method of uniaxial compression test for the quality requirement differs from this study in two aspects: one is the specimen size and the other is the specimen preparation. The specimen for the quality requirement has the same diameter as this study, 100 mm but the height is 127 mm, giving a height-diameter ratio of 1.27 which is much smaller than 2.0 in this study. It is well-known in geotechnical engineering community that, the lower the ratio, the larger the strength is. In the specimen preparation for the quality requirement, the compacted specimens are cured in the air for 13 days at 20°C, then immersed in the water at 20 °C for 24 hours. This is considered to be a favorable condition for HMS, which may accelerate the hydraulic reaction in the material, exhibiting a larger strength at a shorter period of curing.

It is seen in Figure 4 that there is a variation in uniaxial compressive strength at a given curing time for each HMS. Moreover, the uniaxial compressive strength for three HMS shows a wide distribution at each curing time. Taking a closer look at the data, it seems that: the strength data of HMS-2 form the upper bound while those of HMS-1 the lower bound. Here, the ratios of strength in average between HMS-2 and HMS-1 are 3.04, 4.94, 5.21, 2.93 and 1.52 at curing times of 0, 14, 90, 365 and 730 days, respectively.

It is also seen in Figure 4 that a short period of curing results in a significant underestimate of the strength of a specimen cured long regardless of HMS. For instance, the ratio of strength in average between 14-day cured and 730-day cured specimens is 10.17 for HMS-1, 3.14 for HMS-3 and 4.30 for HMS-3.

Figure 5 compares the uniaxial compressive strength of HMS-1 and HMS-2 with the past results. Here, the past results are those of the experimental study from 2007 to 2009 in which uniaxial compression tests were carried out on 95%-compacted specimens of the HMS brought in from the HMS-1 and HMS-2 manufacturers (Yoshida and Tanaka 2012). In the figure, regression curves, expressed by the following equation, are also given for reference.

\[
\frac{q_s}{q_{00}} = a_1 + \frac{a_2(t/t_0)}{a_3(t/t_0) + a_4}
\]  

(1)

where \(q_s\) is uniaxial compressive strength (MPa), \(q_{00}\) is unit strength of 1 MPa, \(t\) is curing time (day), \(t_0\) is unit time of 1 day, and \(a_1\) to \(a_4\) are regression constants. The regression constants are summarized in Table 3.
together. Regarding the influence of past Ministry of Construction (95%-compacted) data, past Ministry of Construction data (95%-compacted) are not necessarily smaller than those of the present study (100%-compacted). In Figure 5(a), the past HMS-1 exhibits stronger hydraulic nature than the present HMS-1; that is, the strength ratios in average between the present and past HMS-1 are 2.12 and 1.73 for the curing times of 14 days and 730 days, respectively. This variable hydraulic nature overwhelmed the expected influence of compaction efforts on the uniaxial compressive strength.

Regarding HMS-2, on the other hand, it is seen from Figure 5(b) that the influence of compaction efforts on the strength is blurred by the variation of the strength data, especially at longer periods of curing time. Moreover, the development in uniaxial compressive strength seems to differ in the two HMS: the strength of the past HMS-2 tends to increase further with time, compared with the present HMS-2.

From these observations, it can be said that the development of hydraulic nature varies from one manufacturer to another and also that even if the HMS comes from the same manufacturer, it may be different depending on when it comes.

Now, assuming that the regression results above are applicable to a much longer period of time, say 20 years, one can project the variation of uniaxial compressive strength over 20 years, which is given in Figure 6. It is seen that the strength of the past HMS-1 will reach more than 5.5 MPa at the 20-year curing while those of the present HMS-1 and HMS-2 will settle at about 2.5 MPa. There will be a difference as large as 3 MPa between them.

5 CONCLUSIONS

In this paper, uniaxial compression tests on the specimens cured up to 730 days of the hydraulic graded iron and steel slag (HMS) obtained from three different manufacturers are discussed with a focus on the hydraulic nature inherent in the material. From this, the followings can be pointed out.

- Each HMS exhibited a variation in uniaxial compressive strength for each curing time.

<table>
<thead>
<tr>
<th>Table 3. Summary of regression constants for HMS-1 and HMS-2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>HMS-1 Present</td>
</tr>
<tr>
<td>Past</td>
</tr>
<tr>
<td>HMS-2 Present</td>
</tr>
<tr>
<td>Past</td>
</tr>
</tbody>
</table>
- The strength of three HMS (HMS-1, HMS-2 and HMS-3) was distributed in a wide range for each curing time: the HMS-2 appeared to form the upper bound of the widely distributed strength data, while the HMS-1 the lower bound.
- The increase of uniaxial compressive strength with curing time differed between the three HMS; this point was confirmed by a comparison of the strength data with the relevant past data.
- It would be safe to say that the development of hydraulic nature varies from one manufacturer to another and also that even if the HMS comes from the same manufacturer, it may be different depending on when it comes.

From these findings, it can be speculated that: when two base-course is constructed using HMS, they could exhibit a different durability or service life, even if the thickness and the compaction degree of the base-course are the same between them. In other words, it would be difficult to determine the thickness of HMS base-course rationally to meet a targeted service life. Thus, one would have no choice but to adopt a conservative thickness referring to the existing pavement structures with HMS base-course.

ACKNOWLEDGEMENTS

The author would like to express his gratitude to Messrs. S. Tani, D. Tanaka and T. Furutani, who are all former graduate students, for their conducting most of the experimental work. This research was partly funded by the Nippon Slag Association and the author would like to appreciate it.

REFERENCES

SELECTION OF ROAD SAFETY MEASURES ACCORDING TO CAPACITY, SAFETY AND COST APPROACH

Oğuz SEHTİYANCİ, Kenan KAYACI

DG Turkish Highways, Ankara, TURKEY
e-mail: osehtiyanci@kgm.gov.tr , kkayaci@kgm.gov.tr

INTRODUCTION

Losses and socio-economic impacts due to the traffic accidents, bring about crucial burdens on the countries. The need of an analytic method grows for selection of road safety measures based on operation performance, safety and cost effectiveness.

In this framework, Capacity-Safety-Cost Approach (CSC) makes a major contribution in finding a way out of road safety problems and the prevention of potential road accidents.

In this method (CSC), it is investigated road safety problems, risks and needs, reasons estimated and detected for these needs or problems, targets that are measurable and monitorable for both meeting needs and eliminating problems, do-nothing scenario, alternative solutions that will eliminate underlying causes of road accidents, preferred solution to traffic accidents, realized solutions’ level of success in reaching observable. The monitorable targets are assessed in a systematic way and practical results of this assessment are presented as suggestions for solution to the road safety experts who study in the application area.

SOLVING ROAD SAFETY PROBLEMS: A STRATEGY

Present practice

Traffic problems are often approached from a specific point of view. The road owner (authority, council) identifies a problem, a group of experts develop a solution and the owner takes a decision. Occasionally, involved parties such as residents, school boards, retail associations or councillors are given the opportunity to put their view forward. The danger of a one-sided approach is that not all aspects of the problem are sufficiently dealt with. Was a cheaper solution possible instead of the expensive traffic regulation system? Has the problem actually been solved? (SRD_manual) Are able to give a suitable level of service the selected safety measures? In this point, in this article, emphasis will be on road design in relation to road safety with special attention to design standards, accident analysis and cost-benefit analysis. A step-by-step method (POGSE) and a sub method (CSC) are described to lead from problem recognition to development of adequate and appropriate solutions included accident causes, goals, alternative solutions, capacity and safety performance and cost.

Policies

Road safety policy is in many countries a spearhead action. For instance: it is based on selecting and analysing black spots, giving special attention to vulnerable road users (pedestrians and cyclists) or predominant accident types (speeding, alcohol). In United Kingdom, Sweden, the Netherlands and Denmark this policy has proven to be very successful in reducing the number of accidents and fatalities. However, for continuing the downward trend in fatalities and injuries it is necessary to develop a more comprehensive approach, based on the interaction between humans, vehicles and the infrastructure. In the Netherlands this approach is known as “Sustainable Safety”. In Central and East European countries with high accident rates, due to the strong growth of car ownership and partly inadequate infrastructure, the spearhead policy seems to be the most cost-effective manner to start with.

Approach

The frustrations of interested parties not involved in the process should not be forgotten, nor the interminable discussions afterwards which come too late in the day. A great deal of unnecessary time
and money is wasted in this way, certainly if the situation has to be modified afterwards. Unnecessary because there is a better approach to traffic problems; POGSE. This is a simple aid to quickly and effectively analyse and solve problems. POGSE stands for Problem, Origin (cause), Goal (objective), Solution, Evaluation.

Integrated approach

POGSE is a coordinated approach, integrating a number of logical steps to solve the problems of traffic safety. It promotes consultation and active involvement of all parties concerned (the stakeholders) to systematically seek solutions to traffic problems. The starting point is the opinion that all stakeholders – with their traffic behavior and views on traffic – should play a role in seeking and finding the correct solutions. Communication and cooperation are just as important as traffic science and engineering.

With the POGSE approach all parties involved are assured of the opportunity for maximum input to the decision-making process. The POGSE approach saves time, money and frustration and provides demonstrably better results. In the Netherlands it is applied successfully in various situations, both simple and complex, to solve traffic problems [1].

The POGSE approach has many advantages. Most important, naturally, is the quality of the decision. The broad approach generally generates points of view that are overlooked in the one-sided approach. With the POGSE approach the various points of view can be carefully weighed up against one another.

The approach, simplified by the steps Problem-Origin-Goal-Solution-Evaluation, summaries the entire decision-making process. Contrary to the conventional approach, involved parties are not confronted with ready-made solutions, but they are given the opportunity to participate and react early on in the process.

POGSE: step by step

Problem
A problem is mainly related to a location (junction) or a road link. It can be determined on the basis of accident records, but may also follow from complaints of local residents. Insight is needed in the present and future function of the road or road links.

The trap of confusing the problem itself and the cause of the problem should be avoided. Consensus of the stakeholders on the real problem and the intended function of the road (link) are required before the next step is started.

Origin (cause)
When agreement regarding the nature of the problem is achieved; it is possible to proceed to the following phase: indicating possible causes. Opinions can differ drastically here between the stakeholders. Car drivers, for example, can be inclined to point to irresponsible cyclists’ behavior, while vice versa there are complaints about speeding by car drivers. At this stage, clear, independent research is indispensable. It is essential for all opinions to be considered, as more than one cause can lead to the identified problem. Also with this step, agreement on the cause(s) of the problem is a requirement before proceeding to the next step.

The analysis may concern:
- accidents (black spot analyses, see chapter 10);
- complaints (local residents, drivers, school boards, other pedestrians);
- traffic data (speeds, volumes);
- confusing road lay out;
• evaluation of measures (reconstruction or else) taken in the past (see the last step of the POGSE in this chapter).

Goal (objective)
Once problems and causes have been analysed and established, a common objective needs to be formulated. For example: within a certain period the number of accidents at a junction have to be halved, or cyclists are not to be mixed with fast speeding traffic on a particular road link. In every case, the description of the objective needs to include the highest achievable return.

If an agreeable objective cannot be specified, there is a danger of remaining on a too general level like “Improving the road safety”. Make sure the objective can be measured by defining a quantified improvement. In the evaluation the results of the measures taken (the solution) will be checked or audited against the goals identified in this step. When a specific, common goal is agreed, possible solutions can be identified and implemented, which is the next step.

Solution
This step is to devise possible solutions, in which the traffic expert has an important role. The input or basis for optional solutions are the conclusions of the previous steps (the goal in particular). The stakeholders may propose alternative solutions to facilitate discussions and decisions. The final choice is made considering the following:

• which solutions have the best effect (comes nearest to the goal or goals)?
• what is the cost?
• what is the level of operation performance?
• are other works foreseen to combine with specific measures?

Evaluation
Evaluation is the continuous monitoring of the effects of measures, followed by comparison with the set goals. Monitoring means collection and analysis of traffic data and accident data, complaints. Experience shows that implemented measures do not immediately lead to an improvement of the situation; it may even worsen initially. Evaluation is also very important to gather experience and knowledge about safety measures within certain circumstances. Comparison with the set goals means: an answer to the question whether results are as expected (do the results comply with the goals).

An evaluation period of three years is generally observed before definite conclusions are drawn. If found that the benefits are not satisfactory, the POGSE approach should be repeated, most probably leading to a refinement of the initial solution.

HOW CAN ENGINEERING MAKE THE ROADS SAFER?

Road crashes are almost always multi-factor events involving the driver, vehicle and/or the environment, including the road. There are many elements of the road that contribute to safety outcomes. These include road width, alignment, the presence of and type of intersections, and roadside hazards such as trees, poles and ditches. In the past, road safety has focused on the road user through enforcement and education. It is still crucial that we enforce safe limits on the road system.

However, it is also recognized that whatever we do to make drivers more alert, law abiding and competent, some will still make mistakes. They should not, however, have to suffer unnecessarily harsh crash outcomes, such as serious injury or death. We must work on designing and operating a road network that is more forgiving and protecting of driver mistakes and crashes.

By re-designing roads to make them safer, we can reduce both the number of crashes that happen and the severity of those that do occur. All risk cannot be eliminated through infrastructure and vehicle safety improvements alone. Drivers must always share responsibility for a safe road system.
Engineering measures can influence the messages we receive as drivers by making a road more ‘self-explaining’. Ideally, each type of road should have a recognizable and distinctive set of self-explaining features such as signage, lane width, road markings and speed limits. This ensures roads are predictable so that road users can expect particular safety features on each type of road.

To reduce the consequences of those crashes that do occur, roads can also be made more forgiving. Examples of specific road environment treatments, their potential effect on the four main crash types, and their relative costs are provided in Table 1.

Table 1: Potential reductions (%) in various injury crush types

<table>
<thead>
<tr>
<th>Alternative Solutions</th>
<th>Safety Problem</th>
<th>Cost</th>
<th>Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Treatment</td>
<td>Head-on crashes</td>
<td>Run off crashes</td>
<td>Intersecti on crashes</td>
</tr>
<tr>
<td>Road signs and delineation</td>
<td>25-40</td>
<td>25-40</td>
<td>25-40</td>
</tr>
<tr>
<td>Rumble strips</td>
<td>10-25</td>
<td>10-25</td>
<td>1-5 yil</td>
</tr>
<tr>
<td>Central median hatching</td>
<td>10-25</td>
<td>1-5 yil</td>
<td>$</td>
</tr>
<tr>
<td>Speed reduction (per 10 km)</td>
<td>15-40</td>
<td>15-40</td>
<td>15-40</td>
</tr>
<tr>
<td>Dedicated lanes for turning traffic</td>
<td>25-40</td>
<td>25-40</td>
<td>5-10 yil</td>
</tr>
<tr>
<td>Removal of roadside objects</td>
<td>25-40</td>
<td>5-10 yil</td>
<td>$S</td>
</tr>
<tr>
<td>Roadside barriers</td>
<td>25-40</td>
<td>10-20 yil</td>
<td>$S</td>
</tr>
<tr>
<td>Shoulder sealing</td>
<td>25-40</td>
<td>5-10 yil</td>
<td>$S</td>
</tr>
<tr>
<td>Roundabout</td>
<td>60+</td>
<td>10-20 yil</td>
<td>$S - $$$</td>
</tr>
<tr>
<td>Straighten alignment</td>
<td>25-40</td>
<td>25-40</td>
<td>20+</td>
</tr>
<tr>
<td>Overtaking lanes</td>
<td>10-25</td>
<td>10-25</td>
<td>10-20 yil</td>
</tr>
<tr>
<td>Divided roads and/or median barriers</td>
<td>40-60</td>
<td>40-60</td>
<td>10-20 yil</td>
</tr>
<tr>
<td>Grade separated junctions</td>
<td>40-60</td>
<td>20+</td>
<td>$S - $$$</td>
</tr>
</tbody>
</table>

Note: $ \rightarrow$ Less than $50,000 per km or low cost, $S$ \rightarrow $50,000 to $500,000 per km or medium cost, $$ \rightarrow$ greater than $500,000 per km or high cost

**WHICH QUESTIONS DOES POGSE AND/OR CSC METHOD ANSWER?**

This method(s) provides answers to the following questions:
- Which measures can be used to reduce the number of traffic accidents or the severity of injury in such accidents?
- Which accident problems and types of injury are affected by the different measures?
- What effects on accidents and injuries do the various road safety measures have according to international research?
- What effects do the measures have on mobility and the environment?
- What are the costs of road safety measures?
- Is it possible to make cost-benefit evaluations of the measures?
- Which measures give the greatest benefits for traffic safety seen in relation to the cost of the measures?

Not all these questions are equally easy to answer, and it is not always possible to give a precise or conclusive answer. For example, the effect of a measure on accidents may vary from place to place, depending on the design of the measure, the number of accidents at the spot, any other measures that have been implemented, etc. As a result, different studies of the same measure may provide different conclusions. An attempt has been made to identify sources of variation in study findings and to try to form as homogeneous groups as possible when presenting estimates of the effects of measures on road safety.
While solutions to safety problems are produced, similar tables for effectiveness of safety measures, service life, cost and operating performance effects in different areas, should be prepared. These areas are road design and road equipment, road maintenance, traffic control devices, vehicle design and protective devices, vehicle and garage inspection, driver training and regulation of professional drivers, public education and information, police enforcement and sanctions.

CASE STUDIES
In this chapter, it is only shown engineering solutions.

Case Study 1 For POGSE & CSC Approach [5]

Problem: Intersection crashes are one of the most common types of crash problem, particularly in urban areas. In rural areas, where vehicle speeds are high, the consequence of collisions at intersections can be particularly severe.

The chances of avoiding serious injury or death reduce dramatically above 50 km/h for side impacts for the most modern types of cars, and is far less than this for older vehicles, and particularly for vulnerable road users.

A number of different intersection crash types can occur, including:

- Collision between oncoming vehicles, particularly when one is turning across traffic
- Right-angle collisions, where neither vehicle is turning (often occurring at high speed)
- Right-angle or side-swipe collisions where one or more vehicles are turning
- Rear-end crashes

Origin: There are a number of causes of intersection crashes, including:

- Inadequate sight distance to on-coming vehicles
- High approach speeds
- Lack of intersection visibility (road users are not aware of the intersection)
- Lack of gaps in traffic
- Complex intersection layout
- Poor road surface condition

Goal(s): Reduction casualties 60% in 2 years

Solution(s): Possible solutions are listed below:

Table 2: Possible solutions for problems in case study 1

<table>
<thead>
<tr>
<th>Treatment</th>
<th>Estimated cost</th>
<th>Casualty Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Delineation</td>
<td>Low</td>
<td>10-25%</td>
</tr>
<tr>
<td>Intersection - Delineation</td>
<td>Low</td>
<td>10-25%</td>
</tr>
<tr>
<td>Intersection - Turn lanes (Signalised)</td>
<td>Low to medium</td>
<td>10-25%</td>
</tr>
<tr>
<td>Intersection - Turn lanes (Unsignalised)</td>
<td>Low to medium</td>
<td>10-25%</td>
</tr>
<tr>
<td>Parking improvements</td>
<td>Low to medium</td>
<td>10-25%</td>
</tr>
<tr>
<td>Skid resistance</td>
<td>Low to medium</td>
<td>25-40%</td>
</tr>
<tr>
<td>Speed management</td>
<td>Medium</td>
<td>25-40%</td>
</tr>
<tr>
<td>Railway crossing</td>
<td>Medium</td>
<td>60% or more</td>
</tr>
<tr>
<td>Intersection - Signalise</td>
<td>Medium</td>
<td>25-40%</td>
</tr>
<tr>
<td>One way network</td>
<td>Medium</td>
<td>25-40%</td>
</tr>
<tr>
<td>Intersection - Roundabout</td>
<td>Medium to high</td>
<td>60% or more</td>
</tr>
<tr>
<td>Traffic calming</td>
<td>Medium to high</td>
<td>25-40%</td>
</tr>
<tr>
<td>Intersection - Grade separation</td>
<td>High</td>
<td>25-40%</td>
</tr>
</tbody>
</table>

If selected measure is Roundabout, the needed information is that a roundabout is a one-way roadway around a circular central island. Entry to roundabouts is controlled by 'give-way' markings and signs. Vehicles already on the roundabout typically have right-of-way. Roundabouts cause little delay in low
to medium traffic flows, and require less maintenance than signalized intersections.

Geometric design is crucial to the safety of a roundabout. Curves on the approaches to require all vehicles to slow down before entering. The centre island layout ensures that traffic moves in a one-way direction and that slow speeds are maintained around and at exits to the roundabout.

The rules governing roundabout use also help to improve safety. Drivers approaching a roundabout need to slow and give way to vehicles already in the roundabout, and be prepared to stop. As a result, roundabouts can virtually eliminate often severe right-angle, left-turn (or right-turn), and head-on collisions.

Benefits will be provided:
- Minimal delays at lower traffic volumes.
- Little maintenance required.
- Crash severity is usually lower than at cross intersections.

Implementation issues are below:
- Solid structures should not be located on the central island.
- High painted kerbs around the island can reduce the risk of it being run into.
- Poor visibility on the approach to roundabouts, or high entry speeds, can lead to crashes.
- Facilities to help pedestrians cross the arms of the intersection should be provided in most urban locations.
- Roundabouts can be difficult for large vehicles, particularly buses, to use.
- Designers should be conscious of the risk that roundabouts can be present for cyclists and other slow vehicles, such as animal drawn vehicles.

Case Study 2 for POGSE & CSC Approach

Problem
In the last 3 years, traffic accidents has occurred with 8 dead, 10 injured and 20 vehicles vehicle damaged in K rotary junction located at intersection of AB main road & CD secondary road.

Other Inputs
- Annual average daily traffic (AADT) is 9,000 vehicles/day on main road and 6,000 vehicles/day on secondary road. It is estimated that after 10 years, AADT will be 15,000 on major road and 9,000 vehicles/day on secondary road.
- The budget is 1 million TL (1/2 million $)

Origin(s)
According to accident analysis, the main reason of accidents are high speed, side collision and run-off accidents.

Goal(s)
Building a safer intersection to prevent deaths within 6 months.

Proposed solution(s)
1. Increasing horizontal marking and vertical signing in existing intersection (estimated cost is 50,000 TL/25,000 $)
2. Elimination of vision problems on existing intersection (100,000 TL/50,000 $)
3. Building a new signalized intersection (250,000 TL/125,000 $)
4. Building a new roundabout (600,000 TL/300,000 $)
5. Building a new grade separated junction (3,000,000 TL/1,500,000 $)

Evaluation process and selection of suitable solution
- In this case study, it is predicted that weightiness of safety (S) is 40%, capacity (C) is 30% and cost (C) is 30%. (Note: The weightiness of safety, capacity and cost have to be determine the type of need. Shortly, if your need additional capacity, then, weightiness of capacity has to be higher than others).
• Then the supply level of needs of solutions have measured in terms of operating performance (capacity) and safety.
• The last step for this process is ranking.

Table 3: CSC (capacity, safety and cost) effects of proposed solutions for the problem in case study 2

<table>
<thead>
<tr>
<th>No</th>
<th>Proposed solutions</th>
<th>Safety (40%)</th>
<th>Capacity (30%)</th>
<th>Cost (30%)</th>
<th>Total score</th>
<th>Implementation of priority</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Increasing horizontal marking and vertical signing on existing intersection</td>
<td>5</td>
<td>5</td>
<td>30</td>
<td>40</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>Elimination of vision problems on existing intersection</td>
<td>5</td>
<td>5</td>
<td>30</td>
<td>40</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>Building a new signalized intersection</td>
<td>15</td>
<td>20</td>
<td>20</td>
<td>55</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>Building a new roundabout</td>
<td>40</td>
<td>30</td>
<td>20</td>
<td>90</td>
<td>1</td>
</tr>
<tr>
<td>5</td>
<td>Building a new grade separated junction</td>
<td>35</td>
<td>30</td>
<td>5</td>
<td>70</td>
<td>2</td>
</tr>
</tbody>
</table>

Note: Firstly, the weight ratio for safety, capacity and cost parameters are determined. The nature of the problem are important while these ratios are determining in Table 3. If the basic problem is related to safety, the safety ratio has to be higher then others. On the other hand, if the basic problem is related to capacity, the capacity ratio has to be higher then others. Then alternative solutions for the problem are identified. For each alternative solutions, the safety, capacity and cost points are scored out of 100 and then multiplied by their ratios. The ranking process is done according to the total score for each alternative solution.

According to POGSE or CSC approach, the most appropriate solution is “Building a new roundabout” and the second solution is “Building a new grade separated junction”

Case Study 3 for POGSE & CSC Approach

Problem (Need): In determination of junction type, there is a need for an analytical method that is combined capacity, safety and cost parameters in Turkey. Therefore, a working group of geometrical standards has been established within the body of Turkish Road Association in Turkey1.

Solution: In this study, while the type of a planning junction is being determined, a model with two phases have constituted to balance safety, capacity and cost:

• Phase 1: This phase constitutes 6 steps. As seen Table…, design speed, volume/capacity ratio, at grade intersection density, accident severity index, density of public transport stops and density of pedestrian crossing are evaluated on every steps. According to weightiness of step, GS (grade separated junction) and/or AG (at grade intersection) score is determined.
• Phase 2: If total AG score is higher than total GS score, then it is used Figure… for the type of AG. In the contrary case, it is used Figure… for the type of GS.

Table 3: Determination process of junction types in case study 3

<table>
<thead>
<tr>
<th>Weightiness (20%)</th>
<th>Design speed (km/h) (Step 1)</th>
<th>Volume / capacity ratio (Step 2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>110</td>
<td>Main road</td>
<td>&lt;0.6</td>
</tr>
<tr>
<td>90</td>
<td>Secondary road</td>
<td>&gt;0.6</td>
</tr>
<tr>
<td>70</td>
<td>≤50</td>
<td></td>
</tr>
<tr>
<td>110</td>
<td>GS</td>
<td></td>
</tr>
<tr>
<td>90</td>
<td>GS-AG</td>
<td></td>
</tr>
</tbody>
</table>

http://www.ytmk.org.tr/CgrupDetail.asp?ID=69
<table>
<thead>
<tr>
<th>Weightiness (20%)</th>
<th>At grade intersection density (number/km) (Step 3)</th>
<th>Weightiness (10%)</th>
<th>Accident Severity Index (Step 4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main road ▶</td>
<td>&lt;1.74 1.75-2.99 &gt;3</td>
<td>Main road ▶</td>
<td>≥45   15-45 &lt; 15</td>
</tr>
<tr>
<td></td>
<td>GS    GS-AG AG</td>
<td></td>
<td>GS    GS-AG AG</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Weightiness (10%)</th>
<th>Density of public transport stop (number/km) (Step 5)</th>
<th>Weightiness (10%)</th>
<th>Density of pedestrian crossing (number/km) (Step 6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main road ▶</td>
<td>&lt;1.74 1.75-4.99 &gt;5</td>
<td>Main road ▶</td>
<td>&lt;1.74 1.75-4.99 &gt;5</td>
</tr>
<tr>
<td></td>
<td>GS    GS-AG AG</td>
<td></td>
<td>GS    GS-AG AG</td>
</tr>
</tbody>
</table>

Figure 1: Capacity limits for AG
Figure 2: Flow chart for determination of GS types

CONCLUSIONS

Data relevant to road safety are collected from a number of different sources including police reports and hospital admissions. This data can then be coded and entered on to a computerized database system.

Summary information on deaths and injuries (and in some instances, non-injury crashes) can be used to determine the scale of the safety problem in an area, country or region. However, more detailed information is required in order to determine the causes of crashes, and from this, the types of solutions that might be applied to address these problems. A variety of information is typically collected in the event of a crash. This might include location details, severity, driver factors (such as age of driver and passengers), vehicle factors, road environment factors (e.g. whether the crash occurred at an intersection, the types of road features present, weather conditions etc.) and contributory factors to the crash (e.g. speed, alcohol).

Examination of this detailed information can help identify key factors in crash causation, information that is critical in planning road safety actions. Details of crashes at specific locations can also be used to plan engineering based solutions and enforcement initiatives [6].

REFERENCES
1. DHV Environment and Transportation (2005), Sustainable safe road design A practical manual, file W0937-01.001, Registration number MV/SE2005.0903, version 5
THE PREDICTION OF PAVEMENT FRICTION FROM TEXTURE – PITFALLS AND POTENTIAL

Paper prepared for the
International Road Federation World Meeting

Bali, Indonesia
November 17-19, 2014

By:

D.J. Swan, P.Eng., M.Eng.
Fugro Roadware
2505 Meadowvale Blvd.
Mississauga, ON L5N 5S2
Canada
djswan@fugro.com
+1(905)567-2780

Dr. Nima Kargah-Ostadi
Fugro Roadware, Inc.
8613 Cross Park Dr.
Austin, TX 78754
U.S.A.
nkargah-ostadi@fugro.com
+1(512)977-1800
# TABLE OF CONTENTS

Abstract ........................................................................................................................................... 2  
Introduction ..................................................................................................................................... 2  
Measuring Pavement Texture ......................................................................................................... 2  
  Sand Replacement Test (ASTM E965-96) (5) ............................................................................... 3  
  Circular Texture Method (ASTM E2157-09) (6) ....................................................................... 3  
  Outflow Meter Test (ASTM E2380-09) (7) .................................................................................. 3  
  Mobile Collection Using Single Point Laser Profilers (ASTM E1845-09) (8) ......................... 3  
  Mobile Collection Using 3D Scanning Lasers ............................................................................. 4  
Measuring Skid Resistance ........................................................................................................... 4  
  Skid Resistance Using the Locked Wheel Device (ASTM E274-11) (13) ............................ 5  
  Sideway-force Coefficient Routine Investigation Machines (BS 7941-1:2000) (14) ............. 5  
  Fixed Slip Friction Tester (BS 7941-2:2000) (15) ..................................................................... 5  
  British Pendulum Tester (AASHTO T 278 & ASTM E303) (16) (17) ................................. 6  
  Direct Friction Tester .................................................................................................................. 6  
Relating pavement texture data to skid resistance ........................................................................ 6  
Case Study – Louisiana Department of transportation and Development (DOTD) ...................... 8  
Conclusions and Recommendations ............................................................................................. 13  
References ...................................................................................................................................... 14
ABSTRACT

Reduced pavement friction under wet conditions is known to be a major safety concern. It is difficult to quantify the number of vehicle incidents that are directly or indirectly caused by friction properties of the pavement’s surface, however numerous research initiatives have investigated the increased risk of vehicle incidents under wet surface conditions. The ability of the pavement surface to allow vehicles to stop safely is one of the few design factors controlled by highway agencies. As such, numerous methods are available to measure the tire friction on the pavement surface. The industry has also spent time looking to relate the surface macrotexture, which can be measured cost effectively, to help predict other friction collection equipment results, which can be slower and more expensive to collect due to the use of water and slower operating speeds. This paper describes the state of the industry on collection of this information. It will also provide a clear understanding of how and why to measure both friction and texture, a discussion of the relationship and interaction between friction and texture and the knowledge needed to determine when to consider friction testing and how to determine candidate sections.

INTRODUCTION

Highway designers are always looking for ways to improve highway and road safety with the factors that they can control. Reduced pavement friction under wet conditions is known to be a major safety concern. It is difficult to quantify the number of vehicle incidents that are directly or indirectly caused by friction properties of the pavements surface, however numerous research initiatives have investigated the increased risk of vehicle incidents under wet surface conditions. A sample of these research findings includes:

- In one before and after study (7) performed across English highway pavements, three studies were investigated whereby the authors reported wet weather and dry weather crashes separately. In two cases, dry road crashes fell by 28% and 21% respectively, while wet weather crashes fell by 63% and 71%. In one case, dry road crashes increased by 16%, while wet road crashes fell by 68%.
- Another study (8) conducted crash and pavement friction data obtained across mostly rural interstates and parkway roadways in Kentucky, USA. An increased crash rate at pavement friction (skid number) values of less than 40 (using a lock wheel friction tester with SN40R tire) were determined for low and moderate traffic levels.

Factors affecting pavement friction include pavement surface characteristics, vehicle operating parameters, tire properties, and environmental factors. There are many issues associated with the measurement of pavement texture and the skid resistance. These factors have led to much research and the development of different equipment and procedures to measure different aspects of the problem.

MEASURING PAVEMENT TEXTURE

Pavement texture has been defined in two general categories, based on the size of the surface roughness. Pavement microtexture is defined by the surface asperities (amplitude deviations) of the individual aggregates, with wavelengths less than 0.5mm. The crystalline structure of the aggregates surface, and/or the fine particles of the surfaces mix provide texture on this scale. Under wet conditions, whereby a water film is present on the road surface, microtexture provides the gritty surface to penetrate water films to provide resistance between the tire and the pavement (1) (2).
Pavement macrotexture is formed by the presence and patterns of grooves in Portland concrete pavements, or by the connection between surface and internal pores in asphaltic concrete surfacing material. Pavement macrotexture, defined as wavelengths between 0.5mm and 50mm, influences the pavement drainage performance and provides the hysteresis forces through the deforming of the tire tread.

While many different measures of texture have been developed over time, some of the most commonly used around the world today include:

- Sand replacement test
- Circular texture method
- Outflow meter
- Mobile collection with point laser
- Mobile collection with 3D laser

**Sand Replacement Test (ASTM E965-96) (5)**

The average depth of the texture below the highest points in the surface is measured by filling the voids with a known volume of sand or glass spheres, followed by spreading of the material into a circular patch so that the material is level with the peaks of the surfacing aggregate. Numerous measurements of the patch’s diameter are then taken to determine the average texture depth, formally known as mean texture depth (MTD). This method of determining macrotexture depth is a simple and cost effective test however; it can be subjective based upon operator experience or inconsistency between tests due to the lack of direction provided in this standard for spreading rates.

**Circular Texture Method (ASTM E2157-09) (6)**

The circular texture meter uses a laser to measure the profile of a circle 284mm in diameter and 892mm in circumference. Texture, here calculated according to the mean profile depth (MPD), is calculated for the eight individual triangles of equal area within the tested circle. The reported MPD of the circular texture meter is the average of these eight segment depths.

**Outflow Meter Test (ASTM E2380-09) (7)**

The outflow meter test is a volumetric test method which measures the water drainage rate through the pavement surface texture (macrotecture) using a cylinder with a rubber ring facing down to the pavement and a sensor to measure the rate of drainage. Through the measurement of the escaped time, the test replicates the hydroplaning potential of a vehicle’s tire. This method however is not able to calculate the MPD or RMS texture of a pavement with good correlation. Further the method is slow, requires lane closures and is only able to sample on small areas of pavement.

**Mobile Collection Using Single Point Laser Profilers (ASTM E1845-09) (8)**

Survey vehicles equipped with single or numerous high frequency lasers (of footprint ~1mm) are typically fitted in either one or both of the vehicle’s wheelpaths, providing the means to collect texture profile data at traffic speeds (in excess of 100 km/h). As the method relies on scattered light to measure texture amplitudes, laser devices are unable to operate effectively under wet surface conditions. Two
texture properties can be calculated using this collection method; the root mean square displacement (RMS) of the texture depth and the mean profile depth (MPD). The RMS represents the texture depth as an ‘average’ displacement about the notional datum level within the plane of the surface. The MPD is a calculation of the average depth below the peaks in the surfacing along the line of the profile, providing a two-dimensional representation of the volumetric sand patch test.

**Mobile Collection Using 3D Scanning Lasers**

The recent development of 3D scanning lasers has provided an additional means of network level collection of pavement macrotexture data. The systems make use of two scanning lasers, each producing 2,000 data points over a 2m width of pavement. This provides sufficiently dense texture data (transverse resolution of 1mm) to determine MPD and a digital model of the sand patch test (ASTM E965) in each of the five standard AASHTO pavement bands (centre, left and right wheel path and outside bands). The road porosity index (RPI), defined as the volume of sand divided by the area it occupies, is used to determine the sand patch equivalent. The advantage of 3D scanning lasers is their ability to measure macrotexture across the entire pavement width at traffic speeds. However its current limitation is in its operating frequency (up to 5,600 profiles per second), limiting its longitudinal resolution to 5mm or 2.5mm at vehicle speeds or 100 km/h and 50 km/h respectively.

**MEASURING SKID RESISTANCE**

Pavement friction is caused by two principal frictional forces working together – namely adhesion and hysteresis. Adhesion is the frictional force between the immediate surface of the pavement and tire through bonding and interlocking mechanisms (affected by microtexture). Adhesion is at its greatest when a smooth tire is in contact with a smooth pavement surface, maximising the total contact area, however with the presence of water (acting as a lubricant) these bonds are substantially broken, reducing the available microtexture (5). Hysteresis, known as the resistance to deformation and recovery of the tire surface by surface aggregates, is highly dependent upon the aggregate’s shape and placement across the road surface to effectively deform the tire (affected by macrotexture). While hysteresis is unaffected by the presence of water, good macrotexture performance (efficient drainage capability) will improve adhesion properties of the aggregate’s surface. (10)

Mayer and Kummer (6) emphasize that it is not possible to observe adhesion and hysteresis separately except in the case of a clean, dry surface plate where there is so little variation that hysteresis can be ignored, or whereby the surface is so well lubricated that adhesion properties become negligible.

Test equipment currently used for the measurement of pavement friction becomes complicated as friction is known to vary through the braking cycle. The equipment (typically portable or trailer mounted) has been designed to measure the horizontal friction and vertical load on the tire or pad continuously during testing. The following common test methods are known to be widely used within the industry:

- Locked Wheel
- Sideway-Force Coefficient
- Fixed Slip
- British Pendulum Tester
• Direct Friction Tester

![Diagram of tire and surface aggregates](image)

**Figure 1. Interaction forces between vehicle’s tire and surface aggregates (4)**

**Skid Resistance Using the Locked Wheel Device (ASTM E274-11) (13)**

The resistive drag force and wheel load applied to the pavement are recorded to calculate the coefficient of friction of a pavement surface while travelling at numerous speeds. This coefficient of friction is then used to determine the friction number (FN) or skid number (SN) of the pavement section. This trailer mounted device can operate smooth and or ribbed surfaced tires for the successive measurement of the macrotexture and a combination of the macrotexture and microtexture respectively. Benefits of the test include the ability to perform skid testing at slow, moderate and high speeds, allowing for network level operation. Limitations of the system include intermediate friction testing and ability to only survey straight sections of pavement. (1)

**Sideway-force Coefficient Routine Investigation Machines (BS 7941-1:2000) (14)**

A freely rotating wheel, mounted on the vehicle's passenger side, fitted with a smooth rubber tire and angled at 20° to the direction of travel of the vehicle, is applied to the road surface under a known vertical load. A controlled flow of water is dispensed onto the road surface immediately in front of the test wheel so that at the pre-determined velocity, the test wheel slides in the forward direction along the surface. The force generated by the resistance to sliding can then be related to the wet skid resistance of the road surface.

**Fixed Slip Friction Tester (BS 7941-2:2000) (15)**

A gearing of the fix slip method is controlled such that the measurements made using the test wheel is always slipping at a set percentage of the vehicle speed. Operating at a constant 14.5% slip, a slip speed of 7.6 km/h is achieved at a towing speed of 50km/h, while the vertical load and horizontal force on the
axle are measured to determine the friction encountered. The single test wheel (smooth tire) of the fixed slip trailer commonly traffics the outer wheel path of the outer lane so that lane closures are not required. When operating the device in continuous operation mode, large volumes of water are required. Consequently this device is best used for post-incident investigations rather than network length surveys. (10)

**British Pendulum Tester (AASHTO T 278 & ASTM E303) (16) (17)**

The British Pendulum Tester (BPT) is static friction device which utilizes a low-speed swinging pendulum apparatus, fixed to a standard rubber pad. The return elevation of the swing arm determines the surface texture (microtexture) properties of the pavement. The apparatus is portable and accurate such that it can be used for project level pavement texture surveys or for laboratory testing purposes. Its downsides include its requirement for frequent calibrations and slow rate of testing. (1)

**Direct Friction Tester**

A horizontal spinning disc (284mm in diameter) is lowered to make contact with a pavement sample or road surface following the spraying of water in front of the device’s rubber sliders. Upon contact the disc speed is gradually reduced due to frictional forces, at which the resistive force (torque) is measured to determine the coefficient of friction (CoF). This stationary test method is capable of simulating slip speeds upwards of 90 km/h at a slip ratio of 100%. The larger contact area of this device compared to the BPT allows greater pavement areas to be tested, however the stationary nature of this device dictates that it is not well suited to network level friction surveying. Similar to the BPT, testing the device on trafficked routes requires lane closures, posing a higher safety risk for operators.

**RELATING PAVEMENT TEXTURE DATA TO SKID RESISTANCE**

Roe at. el. conducted testing regarding the influence of texture depth on high and low speed skid resistance. Sensor measured texture depth (SMTD) averaged over 100m sections were measured alongside friction testing using the SCRIM (sideway force coefficient routine investigation machine) for low speed testing, and the locked-wheel tester for a range of test speeds (20 km/h, 130 km/h) (9). A total of 2,000 skid tests were performed across 133 sites at 22 different locations, testing a range of pavement surfacing types encountered across the UK trunk network. A comparison was made between SCRIM skid resistance results performed at 50 km/h vehicle speed and texture collected using a vehicle mounted sensor. As shown in Figure 2 on the next page, very little correlation resulted between SMTD and skid resistance when all tested pavement surface types are considered.
Meegoda et. al. (10) also conducted correlation testing between Mean Profile Depth (MPD) texture collected using vehicle mounted laser sensors (Selcom 62.5 kHz laser according to ASTM E1845 (8)) and skid number collected using a locked wheel pavement friction tester (according to ASTM E274 (13)). Five asphalt pavements were tested that were each less than five years of age. Average and standard deviation of MPD values were calculated for every 0.1 mile segment, while the average and standard deviation of three skid tests performed using a SN40S (smooth tire test at 64 km/h) tire were also determined for each segment. If the standard deviation of the texture and three skid tests for each segment were below allowable thresholds, the average of each was incorporated into the correlation study. Results of the study (shown in Figure 3) suggested that no positive correlation between MPD and SN40S was evident. Peak SN40S results occurred at an MPD of 0.75mm, following with a decreasing skid number with increasing MPD values until MPD reached 1.1mm. Beyond 1.1m, MDP SN40S skid results were found not to change.

A similar study by Jackson (11) had a peak SN40S value occurring at MPD texture depth of 1.3mm, with a flat response curve that was similar to that discovered by Meegoda et. al. following this with the exception of SN40S results associated with MPD texture greater than 3.81mm found to increase.
While not investigated as a part of this network level study, Meegoda et al. (10) further reported that the water film thickness is a significant contributing factor that should be included in a correlation attempt between skid resistance and pavement texture. A study completed by Persson et al. (12) investigated the influence of water film thickness on rubbed friction at low velocities on dry and wet rough substrates. It was observed from this testing that friction testing along wet surfaces were 20-30% lower than along the corresponding dry surfaces. Through using fluid film lubrication theory, Persson et al. proved this loss in friction could not be completely related to hydrodynamic effects. Persson’s proposed explanation was based upon the sealing of water pools by the rubber where by regions of substrate fill with water could not escape.

CASE STUDY – LOUISIANA DEPARTMENT OF TRANSPORTATION AND DEVELOPMENT (DOTD)

Network wide pavement macrotexture and skid resistance data has been collected across Louisiana Interstate and state highways by Fugro. An Automated Road Analyzer (ARAN) collected continuous pavement macrotexture data using a high frequency laser sensor (Selcom 62.5 kHz laser) located in the right wheel path of the vehicle. The ARAN’s equipped inertial GPS navigation system (see Figure 5) provided the location referencing data needed to align collected texture data with skid test data collected using a locked-wheel skid trailer.

Pavement macrotexture, defined as the mean profile depth (MPD), was calculated from raw laser measurements in accordance to standard ASTM E1845 (8) (shown in Figure 4 on the next page).
Figure 4 Calculation of Mean Profile Depth (MPD) as defined in ASTM E1845-09 (12)

> PAVEMENT DISTRESS
With the ARAN’s LPRAS subsystem, 3D profile data is used for automated detection and full lane width image display using pavement cameras.

> MOUNTING - GPS
Every ARAN is equipped with a GPS and is integrated with other subsystems so that if the receiver cannot lock on enough satellites to determine its position, the ARAN DMI and the ARAN Inertial Reference System will fill in the gaps.

> RIGHT-OF-WAY VIDEO
The ARAN can be outfitted with as many as six PHTV cameras which captures right-of-way images allowing you to virtually view the road from the comfort and safety of your office.

> RUTTING
The Laser Transverse Profiler uses dual scanning lasers to accurately measure the transverse profile of the road with 3268 points over 4 meters.

> ROUGHNESS
The Laser SDP is a longitudinal profile measurement system that provides road profile data capture and real-time roughness index calculation using a combination of high-speed lasers and accelerometers.

> TEXTURE
Smart Texture utilizes high frequency lasers to measure the mean profile depth of road surface microtexture.

> POSITIONING - DMI
The Distance-Measuring Instrument measures ARAN chaining and linear distance travelled. Every ARAN is equipped with a GPS and is integrated with other subsystems so that if the receiver cannot lock on enough satellites to determine its position, the ARAN DMI and the ARAN Inertial Reference System will fill in the gaps.

Figure 5 Survey vehicle equipped customized for LA DOTD data collection

The locked wheel skid trailer performed friction tests across the network at approximately 0.1 mile increments or less, interchanging between smooth (ASTM E524-08 (21)) and ribbed (ASTM E501-08 (22)) tires in the right and left wheel paths of the outer most lane respectively. Skid resistance testing
using the locked-wheel trailer (see Figure 6 below) was performed at 30 mph (48 km/h), 40 mph (64 km/h) and 50 mph (80 km/h). Each wheel of the trailer was equipped with a transducer to measure the vertical and horizontal force experienced by the wheel. The trailer was also equipped with water dispensing nozzles, which spray a controlled amount of water on to the pavement ahead of the test to simulate wet weather conditions. The test wheel is locked at this time and the friction force experienced by the wheel during this action is measured and recorded by the system’s data collection computer. The Skid Number (SN) is calculated to be the force, required to slide the locked wheel at the test speed, divided by the effective wheel load and multiplied by 100.

![Figure 6 Locked wheel skid testing trailer and vehicle](image)

For the purpose of this study, 21 test sections (each of length between 4.0 and 18.5 miles) comprised of asphalt concrete or chip seal surfacing were chosen from Louisiana District 4 highway pavements for correlation between macrotexture and skid data. These 21 sites were selected to ensure a range of macrotexture depths were incorporated into the analysis. Pavement macrotexture data, in the form of Mean Profile Depth (MPD), was averaged across each 0.1 mile segment for comparison against skid data collected at 0.1 mile increments. Only skid data collected at a vehicle speed of 64 km/h was used for the purposes of this study.

Correlation studies were performed at two levels for each of the smooth and ribbed tire types. Firstly data summarized at 0.1 mile increments for all 21 sites were combined to determine the correlation, if any, between macrotexture and skid data. Secondly, a higher level summary was performed whereby section length averages of macrotexture and skid data was made to assess the correlation on a site by site basis. One of the limitations of the study was that macrotexture data was only collected in the outer right wheel path, which made correlation analysis between texture data located in this wheel path and skid data collected in the left wheel path less than optimal.
For the first correlation study, SN data for each respective tire type and the corresponding MPD, averaged over the 0.1 mile section, were utilized. This resulted in a total of 208 data points for the ribbed tire comparison and 198 data points for the smooth tire comparison. While a positive, linear correlation was evident between both tire types and MPD, a very low slope of 1.05 resulted for the ribbed tire. Further, the goodness of fit (R-squared) was low for the smooth tire (Figure 7), with an R-Squared of 0.30, and statistically insignificant for the ribbed tire (Figure 8), with an R-Squared of 0.01.

![Figure 7. Granulated comparison of Skid Number (Smooth Tire) and Mean Profile Depth](image1)

![Figure 8. Granulated Comparison of Skid Number (Ribbed Tire) and Mean Profile Depth](image2)
For the second correlation study, using averaged SN and MPD of entire section lengths of between 4.0 miles (6.4 km) and 18.5 miles (30 km), a near statistically significant (R-Squared value of 0.72), positive, linear correlation (see Figure 9) resulted between the SN using a smooth tire and its corresponding MPD (Figure 9).

![Section Averages - Smooth Tire - Asphalt Concrete](image)

**Figure 9. Section average correlation of skid data using a smooth tire (SN64S) and macrotexture (MPD)**

For the same section length averages, no apparent relationship could be developed between SN based on the ribbed tire and MPD, where a statistically insignificant (R-Squared value of less than 0.01) correlation was again found using this tire (Figure 10). It should be noted that the ribbed tire was restricted to the left wheel path of the skid trailer as opposed to the single point texture laser being restricted to the right wheel path. This could influence the correlation should any expected discrepancy occur in the rate of texture and skid resistance loss between the two wheel paths.

![Section Average - Ribbed Tire - Asphalt Concrete](image)

**Figure 10. Section average correlation between skid data using a ribbed tire (SN64R) and macrotexture (MPD)**
Table 1 compares the linear relationship and regression statistics between correlations performed on the granulated (0.1 mile averages) and section averages for the ribbed and smooth tires.

<table>
<thead>
<tr>
<th>Dependant Variable</th>
<th>Independent Variable</th>
<th>Sample Size</th>
<th>Slope Coefficient</th>
<th>Standard Error</th>
<th>P-Value</th>
<th>R-Squared</th>
<th>Adjusted R-Squared</th>
</tr>
</thead>
<tbody>
<tr>
<td>SN, smooth Tire (Granulated)</td>
<td>MPD</td>
<td>198</td>
<td>9.08</td>
<td>0.98</td>
<td>4.68E-17</td>
<td>0.30</td>
<td>0.30</td>
</tr>
<tr>
<td>SN, smooth Tire (Section Average)</td>
<td>MPD</td>
<td>21</td>
<td>10.06</td>
<td>1.49</td>
<td>2.50E-6</td>
<td>0.72</td>
<td>0.70</td>
</tr>
<tr>
<td>SN, ribbed Tire (Granulated)</td>
<td>MPD</td>
<td>208</td>
<td>1.05</td>
<td>0.89</td>
<td>0.24</td>
<td>0.01</td>
<td>&lt; 0.01</td>
</tr>
<tr>
<td>SN, ribbed Tire (Section Average)</td>
<td>MPD</td>
<td>19</td>
<td>0.03</td>
<td>2.17</td>
<td>0.99</td>
<td>&lt; 0.01</td>
<td>&lt; 0.01</td>
</tr>
</tbody>
</table>

The statistics suggest an improved fit of data to each of the estimated linear relationships for the smooth and ribbed tires, when the data is averaged over longer (section) lengths. It is believed that through performing this averaging of SN and MPD data, variation in SN for a given mean profile depth (MPD) is much reduced, such improving the sum of squares regression.

No relationship between the ribbed tire and MPD could be identified in this research. It is believed that the underlying reason for this is that the profile texture data measured using the laser is unable to replicate the draining of surface water that is typically performed by the ribbed tire. Using section averaged MPD and SN data using the smooth tire, a fair to good regression between MPD and smooth tire friction resulted from this research.

Where entire sections are known to be of consistent surfacing composition and age, such as for rural state and interstate highway sections, an approach where the MPD could potentially be used as a tool for the improved locating of skid resistance testing using a locked wheel device with a smooth tire.

**CONCLUSIONS AND RECOMMENDATIONS**

While much research continues to be done on the relationships between texture and skid resistance, there are little conclusions to be made without more pavement details. Specifically, the influence on factors such as asphalt mix type is often at times difficult to properly identify and develop specific correlations. Since both open graded and dense graded mixes have been used to provide adequate surface friction for many years, the differences in expected texture cannot be readily removed from the measurements.

Continued research is recommended to find a cost effective method to test skid resistance without dependence on factors such as testing speed and surface type. Continued improvement to skid resistance test methods are also recommended to develop tests that can be performed at a larger range of speeds to
improve safety and with reduced dependence on large water sources to increase testing rates and the cost effectiveness of the testing.

Better methods to measure the microtexture of roads at highways speeds may also improve the quality of correlations between texture and skid resistance. As technology improves, it may be possible to better measure the very small fluctuations in the aggregate surface. This may better describe the impact of the tire surface interaction under wet conditions such that it can be tested at a network level.

REFERENCES


ABSTRACT

With some 31,000 road crash fatalities annually and a fatality rate per population exceeding 12.5 per 100,000, Indonesia faces a substantial road safety challenge. Indonesia is in a critical position in moving to stamp control on the levels of safety experienced by its citizens on its road networks. Major change is required. In response to the UN Decade of Action for Road Safety, the National Road Safety Master Plan has been declared in 2011, followed up by the related Presidential Instruction in 2013, committing the Country to the decade of action for road safety. This instruction is also designed to main-stream road safety within agencies and it responds to insufficient community awareness about the serious extent of daily road related trauma. This paper discusses (a) the structure of the national road safety plan in Indonesia; (b) how to identify and implement activities across the sectors by relevant institutions which will deliver the actions specified in the Master Plan, as reinforced with specific responsibilities prescribed in the Presidential Instruction; and (c) a number of key issues that must be tackled and how to resolve these to gain improvement in performance.
Road Safety Policies in Indonesia - The Decade of Action for Road Safety 2011-2020

Bambang Prihartono¹, Eric Howard², Tri Tjahjono³
¹Director of Transportation Ministry National Development Planning/National Development Planning Board
Email for correspondence: bambang@bappenas.go.id
²INDII-AusAID Consultant Principal Consultant of Whiting Moyne – Australia
Email for correspondence: eric.howard@bigpond.com
³Department of Civil Engineering Universitas Indonesia Depok 16424
Email for correspondence: tjhjono@eng.ui.ac.id

INTRODUCTION

This paper addresses the development of road safety policies in Indonesia in response to the United Nations’s Decade of Action for Road Safety 2011-2020. In response to Road Traffic and Transportation Act No 22 year 2009 mandate, the Government of Indonesia (GoI) proclaimed the 2011-2035 National Road Safety Master Plan (the RUNK Jalan) in 2011. This RUNK is also in line with the United Nations declaration of the Decade of Action for Road Safety 2011-2020. The Plan was published in 2011 and sets out an ambitious program for improved road safety performance through to 2035, in 5 year targeted groups of activities. It is soundly based upon the safe system approach which is recommended by the UN in its Decade of Action Plan. This instruction was designed to main-stream road safety within agencies and it responds to the insufficient community awareness about the serious extent of daily road related trauma.

It proposes a reduction in the rate of fatalities per vehicle (and per population) of 50% by 2020 and 80% by 2035, using 2010 fatality levels as the 2010 baseline figure (RUNK, 2011). Implementation of the Plan was not officially budgeted by GoI, but continues to provide a most useful guide for the short and medium term for all stakeholders. In 2013, Presidential Instruction (Impres) No. 4/2013 was decreed by the President of Indonesia and sets out specific actions for the Decade of action for Road Safety (reflecting closely the actions in the RUNK) with responsible agencies identified and timelines allocated by each year for each action. The Impres 4/2013 then needs to be cascaded to sub-national agencies, and provide guidance for a possible realignment of some time frames with the Indonesian national strategic plan (RENSTRA).

The road safety situation in Indonesia is a serious problem. The number of fatalities in 2010 was 31,234 and a number of studies believe fatality levels are still under reported (INDII 2010). Indonesia is in a critical position in moving to stamp control on the levels of safety experienced by its citizens on its road networks. Indonesia also faces high growth of motorized vehicles, in particular, motorcycles. Table 1 and 2 show the growth of motorized vehicles and road safety conditions respectively and Figure 1 shows projections of road crash fatalities in Indonesia for “do nothing” and “do something to meet the target reduction of 50% by 2020 and 80% by 2035” (Yahya et al, 2011). The “do nothing” model was developed by employing Smeed’s Model as described in Jacobs and Sayer, 1983), and the “do something” scenario is based on the target provided in the RUNK.

Table 1. Growth of motorised vehicles in Indonesia

<table>
<thead>
<tr>
<th>Year</th>
<th>2010</th>
<th>2011</th>
<th>2012</th>
<th>2013</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passenger Cars</td>
<td>8,129,091</td>
<td>8,744,731</td>
<td>9,524,666</td>
<td>12,227,325</td>
</tr>
<tr>
<td>Buses</td>
<td>1,071,718</td>
<td>1,986,514</td>
<td>1,945,288</td>
<td>2,348,923</td>
</tr>
<tr>
<td>Goods Vehicles</td>
<td>3,268,586</td>
<td>4,310,046</td>
<td>4,723,315</td>
<td>6,009,188</td>
</tr>
<tr>
<td>Others</td>
<td>265,407</td>
<td>270,152</td>
<td>280,372</td>
<td>349,939</td>
</tr>
<tr>
<td>Total</td>
<td>72,490,059</td>
<td>84,386,226</td>
<td>94,229,299</td>
<td>120,846,027</td>
</tr>
</tbody>
</table>
THE NATIONAL ROAD SAFETY PLAN IN INDONESIA (RUNK) ANSD PRESIDENTIAL DECREE (INPRES) NO. 4/2013

The RUNK is based on principles of sustainability, coordinated and cooperative activity to improve road safety, reflecting an understanding that road safety conditions can only be improved through collaboration by all Indonesians. The first ten year period of this RUNK coincides with the Decade of Action for Road Safety of the Republic of Indonesia 2011-2020 (see Figure 2). The Plan was not officially endorsed by the Government of Indonesia in 2011, but continued to provide a most useful guide for the short and medium term for all stakeholders. In 2013, Inpres 4/2013 was decreed by the President of Indonesia and sets out specific actions (reflecting closely the actions in the RUNK) for the next five years, with responsible agencies identified and timelines set for each year to 2017 with the Ministry of National Development Planning (Bappenas) as the lead agency.

The RUNK reflects the UN Decade of Action Global Plan 2011 - 2020 and its five safe system pillars are drawn from the UN Plan. These are road safety management, safer roads, safer vehicles, safer people and pre and post-crash response. The respective pillar coordinators are the Ministry of National Development Planning/ Bappenas, Ministry of Public Works, Ministry of Transportation, Indonesian National Police and Ministry of Health (see Figure 3). As mentioned above the target of the RUNK is the reduction of road crash fatalities by 50% and 80% by 2020 and 2035 respectively.

An outline description of each of the 5 Pillars is as follows:

**Pillar 1: Road safety management:**
Measures to be developed and implemented under this Pillar are specified in Inpres 4/2013 and include: harmonization and coordination of road safety; Emergency Vehicle Traffic Protocol; road safety research; injury surveillance and an Integrated Information System; a Road Safety Fund; road safety partnership; the development of a Public Transportation Safety Management System; and improvement of road safety regulations. Inpres 4 allocates responsibility for this Pillar to the Minister of National Development Planning (Bappenas) with responsibility including setting of targets and evaluating outcomes.
Pillar 2: Safer roads:
Measures to be taken under this Pillar are specified in Inpres 4/2013 and include: safer road lanes; planning and execution of safer road works and safer road furniture; speed management; implementing improvement in the worthiness of operational standards of safer roads; achieving a safer road environment and achieving safer roadside activities. The Minister of Public Works has responsibility for Pillar 2 which covers the provision of safe road infrastructure through improvement in the planning, design, construction, and operation of roads.

Pillar 3: Safer vehicles:
Measures under this pillar include: providing and improving the periodic test and type test procedures; vehicle speed restrictions; overloading management; elimination of vehicles (scrapping) and achieving a safety standard for public transportation vehicles. The Minister of Transportation has responsibility for Pillar 3, which seeks to ensure that every vehicle used on the road meets safety standards.

Pillar 4: Safer road users:
Measures to be taken under this Pillar include: compliance by operating vehicles; driver condition inspection; driver medical examination; improvement of facilities and infrastructure for driving license (sim) test system; improvement of sim test procedures; technical guidance for driving schools; handling of the five main risk factors; use of electronic law enforcement; formal education on road safety; and conduct of safety campaigns.
The Chief of the Indonesia State Police is responsible for Pillar 4, specifically to improve the behaviour of road users through traffic safety education, to improve the quality of the driving license (SIM) test system and law enforcement on the street, as well as developing a data collection system for traffic crashes.

**Pillar 5: Post crash care:**
Necessary measures to be taken under this pillar are specified in Inpres 4 to include: Pre-Crash Action; Post-Crash Response; guarantee/insurance for accident victims who are treated in referral hospitals; allocation of part of insurance premium for a Road Safety Fund; Pre and Post-Crash Research in handling accident victims. The Inpres 4 allocates responsibility for this Pillar to the Minister of Health through their responsibility for improving pre-accident response management through promotion and improvement of driver’s health in special circumstances/situations and post-accident response management through an Integrated Emergency Management System (SPGDT).

**MANAGEMENT AND OPERATION OF THE RUNK/INPRES4 IMPLEMENTATION**

**Progress summary to date (2013)**
The Inpres 4/2013 is an impressive road safety action plan with significant projects and ambitious timeframes. The Inpres 4 has the majority of the actions scheduled to be delivered in 2013 (44%) and 2014 (20%) – see table below. Given that many of these actions will require multiple years to implement and the Inpres 4/2013 was decreed in April 2013, many of the actions for 2013 have not been completed, however work is in progress on most of these.

In addition, the weighting of timing for the delivery of the actions in the Inpres 4/2013 does not provide sufficient scope for the establishment of road safety as a significant and legitimate agenda item, to provide opportunity for capacity development and “learning by doing”. As illustrated in Figure 4 below World Bank Guidelines show that the strategy of road safety investment starts slowly as capacity and understanding is built prior to a growth phase where a large amount of implementation work can be undertaken. The Inpres 4/2013 has not provided opportunity for this necessary staging of capacity building to occur.

Implementation timelines included in the RUNK (and adopted in the Inpres 4/2013) were, in retrospect, too optimistic and many of the scheduled 2013 actions have not been completed. Realistically these require three to five more years to achieve complete roll out across all of Indonesia. It is suggested that a 20% fatality reduction target by 2020 is more feasible than the planned 50%. (An estimated 30% reduction could be achieved if concerted efforts to install road markings and signage plus infrastructure treatments were introduced with increased enforcement by police which in turn is then supported by the courts.) Essentially, too many enabling actions were required to be put in place in the 2013 timeframe to permit provincial and local government to then implement actions at those levels. Further, the preparation and capacity building of those responsible for delivery at provincial and local government levels would have needed to progress very quickly to attain the stated 50% target by 2020. These targets remain highly unlikely.

There are many 2013 actions where work is in progress but the final outcome is yet to be achieved. Overall implementation status is as follows:
- Five (5) actions: are well advanced or substantially completed;
- Thirty five (35) actions: are underway; and
- Five (6) actions: are to be started.

<table>
<thead>
<tr>
<th>Year</th>
<th>No. of Actions to be completed</th>
<th>% of Actions</th>
</tr>
</thead>
<tbody>
<tr>
<td>2013</td>
<td>38**</td>
<td>44</td>
</tr>
<tr>
<td>2014</td>
<td>17</td>
<td>20</td>
</tr>
<tr>
<td>2015</td>
<td>9</td>
<td>10</td>
</tr>
<tr>
<td>2016</td>
<td>11</td>
<td>13</td>
</tr>
<tr>
<td>2017</td>
<td>9</td>
<td>10</td>
</tr>
<tr>
<td>2018</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>2019</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2020</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>TOTAL</td>
<td>87</td>
<td>100</td>
</tr>
</tbody>
</table>

**Number of Actions carried over (not complete) from 2013**
The following however provide examples of positive progress:

- Police crash data system (Integrated Road Safety Management System) is operational, however further steps are required in terms of technical support and data entry to cover more provinces, and improved data analysis for its full operation;
- National road safety management and coordination arrangements through Bappenas have commenced;
- There is recognition among the agencies that local government, including provincial government, must become a key focus for support and delivery action;
- Universal availability of emergency care for crash injured persons is commencing;
- Directorate General of Highway (DGH) is now operating Blackspot programs, developing training in infrastructure safety technical areas and offering courses to staff in worksite safety management; Road safety engineering training of public works agencies at provincial level by DGH has commenced.
- Directorate General of Land Transportation (DGLT) is reviewing vehicle fitness testing arrangements for heavy vehicles and buses carried out by local government (LG) to support improved performance.

NUMBER OF ISSUES TO GAIN IMPROVEMENT IN PERFORMANCE

Importance Of Effective Road Safety Management
Road safety is about improving a country’s governance and respect for the rule of law. It is not possible to have good road safety performance without good governance. It is not widely understood by the stakeholders that developing workable whole-of-government road safety capacity is in fact a major driver of reform in the public sector of any jurisdiction. So many departments are involved at national, provincial and local levels. Investment in this multi sectoral activity is a key means of introducing change to the effectiveness of the public sector through a focus on cross government cooperation, achieving shared outcomes and accepting accountability. This opportunity should be reflected in the priority and scale of funding given to road safety management investment rather than the small add on amounts that are typically provided for small disconnected projects in the government budget.

The RUNK document refers to a proposed Forum to oversee the management of the national road safety activity within Pillar 1 actions, Road Safety Management. In detail it commits to:

- Establishing a forum/coordinating agency for safety programmes;
- Providing work procedures and governance forums through a coordinating agency;
- Establishing Working Groups on accident response for each pillar; and
- Setting priorities and ensuring effectiveness and sustainability of safety programmes.

Coordination Secretariat in Bappenas
To provide administrative and management support (and selected technical input) to underpin effective collaboration and coordination between the road safety partners it is important to have a permanent secretariat in Bappenas to support the delivery of Indonesia’s RUNK road safety strategy, Inpres 4/2013 (and beyond) action plans and the further development of higher priority actions to strengthen RUNK/Inpres.

The objectives of the Secretariat are:

- Provide oversight of RUNK and key strategic priorities;
- Manage/coordinate road safety management groups;
- Manage and disseminate crash and other road safety data and prioritise research;
- Promote and disseminate best practice guides and information;
- Provide leadership, coordination and facilitation of partnerships;
- Ensure funding and resource allocation for road safety activity; and
- Oversee the legislative program.

In good practice countries the lead agency has the main responsibility within government for managing the country results focus and ensuring that system-wide interventions are agreed and implemented by the responsible authorities across government and wider society. The lead agency concerns itself not only with the development of the national road safety strategy and targets, but also all the institutional management/administrative functions which contribute to its success. This is a critical role for Bappenas as the Secretariat.

Figure 5 shows the proposed management structure for the Forum (Working Group) and Steering Committee, to support coordinated decision making across the national government agencies, plus the necessary provision of support and the establishment of consultative links to enable engagement with other stakeholders.

There is a need at each provincial level (and on a smaller scale for each municipality and regency) for similar management arrangements to those at national level. This will require communication and dialogue with local governments - by the Secretariat - probably through the local Development Planning Board (Bappeda). While police are active in the local government areas with road safety activity, it is important that the provincial and local governments take ownership of the issue locally and convene the local committees with, of course, the full involvement and support of the police. The challenges associated with improving road safety outcomes cannot be regarded as a matter only for police. They need to be part of the team and be closely supported by the local governments.

Figure 5. Proposed Management Structure for National Road Safety

What is notable from initial police analysis of traffic crash data is that most of the reported fatalities from the system are on non-national roads with national roads reportedly contributing less than 30% of all Indonesian fatalities. It is quite possible this will be further revised as data integrity (road status information) improves, but this is what the system is currently indicating. Provincial roads are reportedly contributing 35% of annual fatalities, local (regency) roads 28%, with village roads contributing 7% of fatalities.

Absence Of Funding
Funding levels in Indonesia are totally inadequate. Road safety is receiving paltry resourcing in Indonesia by government at all levels, and relatively low levels of investment. Annually 30,000 Indonesians are being killed on the country’s roads each year. The contrast between and safety, where, for example, The hundreds of millions of dollars invested annually in road infrastructure improvement in Indonesia, contrasts with the inadequate scale of investment in safer road infrastructure and road safety activity more generally.
Given the scale of the governance changes and consequent interventions required at and by all levels of government, ways need to be found urgently to increase funding. Based on high income countries, road safety expenditure on preventative activities by governments in Indonesia would be in the order of some 0.11% of GDP (excluding police costs). This suggests that for Indonesia the basic budget allocation for specific road safety preventative activities (at national and provincial levels) should be in the order of Rp. 6 trillion. There is no evidence that expenditures of this order of magnitude are currently being considered. The Government of Indonesia needs to identify sources of funding to support the early development of road safety capacity. Costs of the road crash epidemic to governments at provincial and national levels are substantial with the Health system advising that up to 46% of admissions to some hospitals are now for injured road crash victims (Basic Health Survey 2013 (Riskesdas) carried out by Ministry of Health, Government of Indonesia).

**Demonstration Projects**

Bliss and Breen (2012) suggested that the demonstration projects are a very important means to set up road safety coordination and consultation activities, i.e. to achieve “learning by doing” on a smaller scale. The generic objectives of demonstration projects and programmes are to:

- Target road safety results in selected high-risk, high-volume roads/areas for long-term and interim periods.
- Provide dimensions for new quantitative target-setting, business cases and roll-out.
- Provide opportunity, focus and mechanisms for policy development and policy pilots.
- Aid institutional strengthening especially lead agency delivery, coordination and multi-sectoral partnership workings, monitoring and evaluation and knowledge transfer.
- Enhance political, professional and public acceptability of important intervention.

The demonstration project proposed in the RUNK Implementation Review Report will be based on the five safe system pillars set out in the RUNK/Impres 4/2013, containing multi pillar corridor activity and including the establishment of road safety management arrangements at provincial level. The demonstration sites would include actions in the following areas:

- Infrastructure safety measures either targeting higher risk corridors with treatments or network wide risk through low cost treatments;
- Social campaigns including the development of content and their delivery;
- Police enforcement preparedness and delivery;
- Improved driver license testing by police;
- Measures to improve safety the condition of trucks and buses including Transport Officer heavy vehicle enforcement preparedness and delivery (with Police and at terminals and weighbridges) including Warrant of Fitness testing for Trucks and Buses at Kota level;
- Road Safety Education delivery in schools, including sustainable teacher briefings;
- Health preparedness for post crash care in corridor; and
- Monitoring & evaluation.

The key deliverables would be improved capacity of the province’s road safety agencies to deliver road safety improvement. It would also provide a clear message to the community that improved performance is achievable.

---

1This estimated figure was determined by Howard for the RUNK implementation study for Bappenas funded by INDI-AusAid in 2014. It is based on levels of road safety expenditure 2004-5 as a proportion of GSP in good practice international jurisdictions (States of Victoria and Western Australia, Australia). (see Bliss and Breen, 2009 pp 259 and 276-277 and ABS, Austra l i a n nat ional statistics 2004-5, http://www.abs.gov.au/ausstats/abs@.nsf/0/4ae650443186d8cf7ca2570b30075af0b7/$file/52200_2004-05.pdf)
CONCLUSION

Bliss and Breen (2012) recommend that for the successful development and delivery of improved road safety performance there are 6 key components:
1. Identify a lead agency in government to guide the national road safety effort.
2. Assess the problem, policies and institutional settings relating to road traffic injury and the capacity for road traffic injury prevention in each country.
3. Prepare a national road safety strategy and plan of action.
4. Allocate financial and human resources to address the problem.
5. Implement specific actions to prevent road traffic crashes, minimize injuries and their consequences and evaluate the impact of these actions.
6. Support the development of national capacity and international cooperation.

Based on the evaluation to date, the direction of road safety policies in Indonesia is on the right path. Since 2013, Bappenas has been operating as the lead agency in Indonesia for road safety improvement actions. The RUNK and Inpres No.4/2013 are the platform for the institutional adoption of responsibility for the five safe system pillars of the UN’s Decade of Action for Road Safety. However, review of capacity for road traffic injury prevention management is still required at provincial level where most delivery will take place (in association with the local National Police). Currently, outcomes from such a review are only available for East Java Province through support provided by INDII-AusAID in 2013. RUNK is also the references for national safety strategy and action plan. The first three components suggested by Bliss and Breen (2012) for successful road safety performance are addressed at national level through the development of the RUNK and Inpres No. 4 and the establishment of Bappenas as lead agency.

The next step is how to progress with the last three components. The Government of Indonesia needs to allocate an adequate road safety budget and develop capacity building to address the problem. Cascading the road safety plan to the local government level is a key issue – as around 70% of road crash fatalities occur on the provincial and local government roads. Implementing specific actions by developing a series of demonstration projects that involve coordinated stakeholder management and action at provincial level will be a necessary next step. Last but not least, support for the further development of national capacity through international assistance will also remain most important.

Acknowledgments
The paper was based on technical assistance to Bappenas for National Road Safety Plan (RUNK) Implementation Study funded by INDII-AusAID. The authors are grateful to all the stakeholders of the Indonesian National Road Safety Agencies for valuable input and suggestion to this study. This paper is based solely on the interpretation of the authors.

References
**PAPER TITLE**
(90 Characters Max)

Joint Research on Carbon Dioxide (CO₂) Emission from Motorcycle Between Indonesia and Japan

**TRACK**

**AUTHOR**
(Capitalize Family Name)

Agah M. MULYADI¹

Researchers

Institute of Road Engineering

Indonesia

**CO-AUTHOR(S)**
(Capitalize Family Name)

Yosuke NAGAHAMA²

Researchers

National Institute for Land and Infrastructure Management

Japan

Shinri SONE³

Head of International Research Division

National Institute for Land and Infrastructure Management

Japan

Samsi GUNARTA⁴

Head of Laboratory/Traffic and Environment Research

Institute of Road Engineering

Indonesia

**E-MAIL**
(for correspondence)

¹agah.muhammad@pusjatan.pu.go.id, ²³ do-kan@nilim.go.jp, ⁴samsi@pusjatan.pu.go.id

**KEYWORDS:**
Carbon dioxide, motorcycle, model, estimation

**ABSTRACT:**

The background is cooperation between NILIM Japan and IRE Indonesia with theme "Environmentally Friendly Transport System, Using Motorcycle". This study raised the topic of CO₂ released by motorcycle.

IRE research on “Correlation CO₂ emissions for motorcycle to the manufacturing, service life, engine displacement and travel speed”. The method of data processing by converting the fuel consumption data to CO₂. The analysis shown that motorcycle with manufacture 2 able to decrease up to 15% CO₂. Motorcycle with engine displacement of 111-149cc is able to reduce CO₂ from 17% to 43%. Meanwhile, the service life of one to six years resulting CO₂ which relatively similar with difference of 15%. The travel speed of 60km/h able to reduce CO₂ up to 62%.

NILIM research on “Estimation of the CO₂ emission effectiveness in the world, by switch motorcycle to 4-wheel vehicle”. Used statistics data are number of owned vehicles, total mileage and annual mileage per vehicle. Assumed scenarios are five scenarios that the 4-wheel vehicle and motorcycle ratios are difference. Japanese theory on traffic capacity is used for the calculations of CO₂ emission. It is evident from the calculations that the reduction of CO₂ emissions resulting from switch from 4-wheel vehicle to motorcycle.
Joint Research on Carbon Dioxide (CO₂) Emission from Motorcycle between Indonesia and Japan

Agah M. MULYADI¹, Yosuke NAGAHAMA², Shinri SONE³, Samsi GUNARTA⁴

¹Institute of Road Engineering (IRE), Bandung, INDONESIA
agah.muhammad@pusjatan.pu.go.id
²National Institute for Land and Infrastructure Management (NILIM), Tsukuba, JAPAN
do-kan@nilim.go.jp
³National Institute for Land and Infrastructure Management (NILIM), Tsukuba, JAPAN
do-kan@nilim.go.jp
⁴Institute of Road Engineering (IRE), Bandung, INDONESIA
samsi@pusjatan.pu.go.id

1. INTRODUCTION

This research is based on the cooperation between the National Institute for Land and Infrastructure Management (NILIM) Japan and the Institute of Road Engineering (IRE) Indonesia with the theme of "Environmentally Friendly Transport System, Using Motorcycle".

By the end of 2012 the population of motorcycle in Indonesia reached 77.7 million units with their average composition on the road reached 82% (Indonesia Motorcycle Industry Association 2012). The number of motorcycles produce CO₂ emissions which have a negative impact on global warming. Global warming occurs because the greenhouse effect of CO₂ in the atmosphere absorb heat energy and obstructed the heat from the atmosphere to the higher surface. This situation lead to the increase of the average of atmosphere temperature of the earth's surface and rising sea levels due to melting of icebergs, which in the end will alter various natural cycles (Bacon et al. 1992).

Whereas 4-wheel vehicles account for the majority of road traffic modes in European countries, the United States, and Japan, motorcycles account for the overwhelming share of road traffic in Asian countries other than Japan and South Korea (IRF. 2010). Although motorcycles pose a higher risk of traffic accidents than do 4-wheel vehicles, they are also considered to deliver various advantages regarding the protection of the global environment and the handling of traffic congestion. However, it is expected that the overwhelming share accounted for by motorcycles in these Asian countries will come to be occupied by 4-wheel vehicles as these countries experience further economic growth. It is therefore possible that the overall global carbon dioxide emissions from road traffic may increase greatly.

2. ACTIVITIES OF JOINT RESEARCH

The joint research raised the theme of "Environmentally Friendly Transport System, Using Motorcycle". The joint research was started in 2011 and it will be finished in 2017 as show in Table 1.

<table>
<thead>
<tr>
<th>Year</th>
<th>NILIM</th>
<th>IRE</th>
</tr>
</thead>
<tbody>
<tr>
<td>2011</td>
<td>Making field test plan on CO₂ emission from motorcycle, electric motorcycle &amp; 4-wheel vehicle in Japan</td>
<td>Study on design criteria of exclusive lane for motorcycle</td>
</tr>
<tr>
<td></td>
<td>Field test on CO₂ emission from motorcycle, electric motorcycle &amp; 4-wheel vehicle in Japan</td>
<td>Hearing from Japanese motorcycle factory</td>
</tr>
<tr>
<td></td>
<td>Hearing from Indonesian motorcycle factory</td>
<td></td>
</tr>
<tr>
<td>2012</td>
<td>Demonstration of how to apply fuel consumption meter method in Japan</td>
<td>Field test on motorcycle emission in Indonesia by idle test method</td>
</tr>
<tr>
<td></td>
<td>Development of fuel consumption measurement for 4-wheel vehicle on Indonesian road</td>
<td></td>
</tr>
</tbody>
</table>

Table 1 Activities of Joint Research
<table>
<thead>
<tr>
<th>Year</th>
<th>NILIM</th>
<th>IRE</th>
</tr>
</thead>
<tbody>
<tr>
<td>2012</td>
<td>Estimation the effectiveness to reduce CO₂ emissions, using Japanese theory on traffic capacity</td>
<td>Model to predict CO and HC from motorcycle emissions</td>
</tr>
<tr>
<td></td>
<td>Publish joint paper in 14th REAAA conference in March 2013 in Kuala Lumpur</td>
<td>Model to predict CO₂ from motorcycle emissions</td>
</tr>
<tr>
<td></td>
<td>Estimation the CO₂ emissions effectiveness in the world, by switch motorcycle to 4-wheel vehicle</td>
<td>Evaluation CO₂ on motorcycle lane</td>
</tr>
<tr>
<td>2014</td>
<td>Publish joint paper in 1st IRF Asia Regional Congress &amp; Exhibition in Bali</td>
<td></td>
</tr>
<tr>
<td>2015</td>
<td>Provide speed sensor, fuel consumption meter and GPS to be installed on motorcycle</td>
<td>IRE received the equipment and conduct collecting data of CO₂ emission for motorcycle on actual road with actual travel speed and actual geometric condition.</td>
</tr>
<tr>
<td>2016</td>
<td>Sharing data to be analyzed together, finding out the relationship between CO2 emission and actual travel speed and actual geometric condition.</td>
<td></td>
</tr>
<tr>
<td>2017</td>
<td>Making paper together and publish in workshop/international conference</td>
<td></td>
</tr>
</tbody>
</table>

In this joint paper, IRE research on “Correlation CO₂ emissions for motorcycle to the manufacturing, service life, engine displacement (cc) and travel speed” while NILIM research on “Estimation of the CO₂ emission effectiveness in the world, by switch motorcycle to 4-wheel vehicle”.

3. CORRELATION CO₂ EMISSIONS FOR MOTORCYCLE TO THE MANUFACTURING, SERVICE LIFE, ENGINE DISPLACEMENT (CC) AND TRAVEL SPEED.

3.1 Method

The data collected in this research is the fuel consumption motorcycles data. Methods of data processing of the fuel consumption by converting from survey data of fuel consumption to CO₂ emissions and calculated using the equation adopted from the Clean Development Mechanism (CDM) AMS-III Methodology. Shown in equation 1. (UNFCCC/CCNUCC, 2006)

\[
EF_{\text{gCO}_2/\text{km}} = SFC \times NVC \times EF_f \tag{Equation 1}
\]

where:
- \(EF_{\text{gCO}_2/\text{km}}\) = Emission factor for vehicles use gasoline fuel (gCO₂/km)
- \(SFC\) = Vehicle fuel consumption Value (g/km)
- \(NVC\) = Fuel calorific value of fossil fuel consumed by the vehicle (J/g)
- \(EF_f\) = Fuel emission factor of fossil fuel consumed by the vehicle (gCO₂/J)

On the Intergovernmental Panel on Climate Change (IPCC) Indonesia has determined the amount of value for NVC in Indonesia in the amount of 42.66 x 103 J/g and 69.3 x 10-6 EF \(f\) CO₂/J. While for SFC values obtained from surveys of fuel consumption in grams per km. The tools used in the fuel consumption survey is a tool measuring instrument chassis dyno and means of fuel consumption. Instrument chassis dyno is a tool used to run the motorcycle wheels on the condition motorcycle is unmoved or stop, so that the simulated motorcycle just like it was driving on the road. Furthermore, fuel consumption is obtained from the results issued by means of fuel consumption in which the results will be automatically printed from fuel consumption meter. Instrument chassis dyno and fuel consumption meter is shown in Figure 1.
The dependent variable in this study is CO₂ in the unit gram CO₂/km. The independent variables used are the variable of motorcycle manufacturing, service life, engine displacement (cc) and travel speed. In data processing, each independent variable was divided into several classifications. The division of the classification are shown in Table 2.

<table>
<thead>
<tr>
<th>No</th>
<th>Independent Variable</th>
<th>Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Manufactures (Brand)</td>
<td>Manufacture 1, Manufacture 2, Manufacture 3</td>
</tr>
<tr>
<td>2</td>
<td>Engine Displacement</td>
<td>≤110 cc, 111-149 cc, ≥150 cc</td>
</tr>
<tr>
<td>3</td>
<td>Service Life</td>
<td>1 year, 2 years, 3 years, 4 years, 5 years, 6 years</td>
</tr>
<tr>
<td>4</td>
<td>Travel Speed</td>
<td>10 km/h, 20 km/h, 30 km/h, 40 km/h</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50 km/h, 60 km/h, 70 km/h, 80 km/h</td>
</tr>
</tbody>
</table>

Based on the motorcycle classification, motorcycle variations tested as many as 54 motorcycles. Furthermore, each motorcycle tested its fuel consumption in accordance with the variations of travel speed started from 10 km/h, 20 km/h, 30 km/h, 40 km/h, 50 km/h, 60 km/h, 70 km/hour, up to 80 km/h, so as to the motorcycle test data obtained 2,592 data. Every travel speed fuel consumption data recorded per 10 seconds. Analysis method of the CO₂ value relationships with independent variables using simple correlation analysis. This analysis is used to determine the relationship between two variables and to determine the direction of the relationship. Value of the correlation between these variables closer to ±1, accordingly the relationship between these variables is getting stronger.

3.2 Analysis
3.2.1 Correlation Analysis of Manufacture vs CO₂

The relationship between motorcycle manufacture and CO₂ emissions are weak, that is equal to 0.016 (strong correlation closer to 1). Motorcycle with manufacture 2 having the largest correlation value and it has negative value i.e. -0.139, therefore the manufacture 2 tend to have the lowest value of CO₂ followed by manufacture 1 and manufacture 3. Manufacture number 1,2 and 3 referring to motorcycles by brand.
Regarding to Figure 2, the manufacture 2 is able to decrease the value of CO₂ emissions up to 15% compared to manufacture 1 and manufacture 3. Manufacture 2 has an average value of 55.49 gram CO₂/km of CO₂ emissions, while manufacture 1 and manufacture 3 have a higher CO₂ emission values, those are 65.78 gram CO₂/km and 65.85 gram CO₂/km respectively.

3.2.2 Correlation Analysis of Engine Displacement vs CO₂

The analysis show that there are weakness relationship between engine displacement and CO₂ emissions that is equal to 0.211 (strong correlation closer to ±1). Motorcycle with engine displacement between 111-149 cc has the highest correlation value and it has negative value to CO₂ emissions i.e. -0.360. It can be conclude that engine displacement of 111-149 cc has a tendency to have the lowest value of CO₂ followed by motorcycle with engine displacement ≤ 110 cc and motorcycle with engine displacement of ≥ 150 cc.

According to Figure 3, the classification of motorcycles with engine displacement of 111 cc-149 cc tends to reduce CO₂ emissions started from 17% to 43% compared with an engine displacement ≤ 110 cc and ≥ 150 cc. Motorcycles with engine displacement of 111-149 cc its CO₂ tends to be lower due to lower fuel consumption compared to ≥ 150 cc motorcycle. While the engine displacement of ≤ 110 cc which is mostly automatic motorcycle is slightly more wasteful in terms of fuel consumption. The average value of CO₂ emissions of 111-149 cc engine displacement by around 44.1 gram CO₂/km lower than engine displacement ≤ 110 cc and ≥ 150 cc, with the average value of 64.20 gram CO₂/km and 77.46 gram CO₂/km respectively.

![Graph 3](image)

**Figure 3.** Correlation of Engine Displacement vs CO₂

3.2.3 Correlation Analysis of Service Life vs CO₂

Based on the correlations in Table 5, there is a weakness relationship between the motorcycle service life and CO₂ emissions that is equal to -0.022 (strong correlation closer to ±1). At the 2 years and 5 years’ service life value has a tendency to reduce emissions that have a value of -0.067 and -0.054 compared to other years of service life. Figure 4 shows that the classification of motorcycle service life of one year to six years of CO₂ emission relatively the same. Service life which does not have CO₂ significant value that is the highest value with the lowest only differs by 15%. This is because the technology used in the last five years relatively the same using injection technology.

![Graph 4](image)

**Figure 4.** Correlation of Service Life vs CO₂
3.2.4 Correlation Analysis of Travel Speed vs CO₂

Based on the correlations analysis there is a quite strong relationship between travel speed and CO₂ emissions, equal to -0.557 (strong correlation closer to ±1). This shows fairly strong relationship and it has negative value. Motorcycle travel speed at 60 km/h has a tendency to reduce emissions significantly with a value of -0.217, while for the travel speed of 10 km/h has a tendency to increase the CO₂ emission value with a value of 0.572.

In accordance with Figure 5, the value of CO₂ on the travel speed of 10 km/h has the highest CO₂ emissions value that is an average of 113.15 gramCO₂/km. Travel speed decreases gradually around 25% on 20 km/h, 30 km/h, 40 km/h, 50 km/h and reaches the lowest point of 60 km/h with an average value of CO₂ emissions by 42.73 gramCO₂/km, then gradually climbed consecutively to the travel speed of 70 km/h and 80 km/h equal to 11%. In the travel speed of 60 km/h the engine performance has been optimized so that fuel consumption may be lower than low and high speed which requiring greater engines performance.

![Graph](image)

Figure 5. Correlation of Travel Speed vs CO₂

4. ESTIMATION OF THE CO₂ EMISSION EFFECTIVENESS IN THE WORLD, BY SWITCH MOTORCYCLE TO 4-WHEEL VEHICLE

4.1 Trial calculation
4.1.1 Trial calculation conditions and procedure

Indonesia was selected as representative of countries in Asian countries with widespread motorcycles ownership. Japan was selected as representative of countries with scant motorcycles ownership. Road traffic modes of motorcycles and 4-wheel vehicles were subjected to trial calculations. Buses and trucks were excluded from analysis. Figure 6 shows the trial calculation procedure. To conduct trial calculations of CO₂ emissions and vehicle kilometers traveled per year, we used formulae that are presented as follows.

\[ \text{CO}_2 \text{ emissions} = \text{Emission Factor} \times \text{Vehicle kilometers traveled per year} \]  
\[ \text{Vehicle kilometers traveled per year} = \text{Mileage per vehicle per year} \times \text{Number of vehicles owned} \]

Scenarios for the conversion of the road traffic mode from 4-wheel vehicles to motorcycles are the scenarios which are near to traffic mode in Japan or Indonesia, the scenario which supposed that the ratio of motorcycle is the same as 4-wheel vehicle, the scenario which supposed that the ratio of motorcycle is not the same as 4-wheel vehicle (see Table 3).

![Diagram](image)

Figure 6. Trial calculation procedure

<table>
<thead>
<tr>
<th>Scenario</th>
<th>4-wheel vehicles</th>
<th>Motorcycles</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scenario 1</td>
<td>100%</td>
<td>0%</td>
<td>Close to the road traffic mode in Japan</td>
</tr>
<tr>
<td>Scenario 2</td>
<td>80%</td>
<td>20%</td>
<td></td>
</tr>
<tr>
<td>Scenario 3</td>
<td>50%</td>
<td>50%</td>
<td>Close to the road traffic mode in Indonesia</td>
</tr>
<tr>
<td>Scenario 4</td>
<td>20%</td>
<td>80%</td>
<td></td>
</tr>
<tr>
<td>Scenario 5</td>
<td>0%</td>
<td>100%</td>
<td></td>
</tr>
</tbody>
</table>
4.1.2 Number of vehicles owned

4-wheel vehicles account for more than 90% of the number of vehicles owned in Japan, indicating that the motorcycles ownership rate is extremely small. By contrast, motorcycles account for more than 60% of the number of vehicles owned in Indonesia. By contrast, motorcycles account for more than 60% of the number of vehicles owned in Indonesia. Thus the motorcycle ownership rate in Indonesia is comparable to that in other Asian countries (see Figure 7). The numbers of vehicles owned in the above five scenarios are shown in Figure 8.

![Figure 7](image1.png)

**Figure 7.** Number of 4-wheel vehicles and motorcycles owned (European Commission 2010, JAMA 2011, IRF 2010, Bureau of Statistics of the Ministry of Internal Affairs and Communications 2010)

![Figure 8](image2.png)

**Figure 8.** Numbers of vehicles in the various scenarios

4.1.3 Mileage per vehicle per year

Mileage per vehicle per year was calculated by dividing the vehicle kilometers traveled per year by the number of vehicles owned. Because vehicle kilometers traveled in Indonesia was unknown, the average value for Asian countries was adopted. It was supposed that the mileage per vehicle per year before conversion would continue even with changes in the types of vehicle owned (see Figure 9). The vehicle kilometers traveled per year in the scenarios is shown in Figure 10.

![Figure 9](image3.png)

**Figure 9.** Mileage per vehicle per year (JAMA 2010, IRF2010)

![Figure 10](image4.png)

**Figure 10.** Vehicle kilometers traveled per year in the scenarios
4.1.4 Changes in traveling speed resulting from the conversion of the road traffic mode

With the conversion from 4-wheel vehicles to motorcycles, the road space originally occupied by 4-wheel vehicles is replaced by that occupied by motorcycles; thus the road occupation area of the vehicles decreases. Consequently, it was expected that average travel speeds would increase because the traffic jam of the road decreased. By contrast, it was expected that average travel speeds would decrease when motorcycles were replaced by 4-wheel vehicles. Therefore, trial calculations were conducted in regard to changes in travel speeds based on the conversion of the road traffic mode as follows (see Table 4):

(1) Because travel speed data in Indonesia were unavailable, traffic volume survey data from the 2010 Road Traffic Census in Japan were used.

(2) By classifying travel speeds into six ranks of 10-km/h units, percentages of vehicle kilometers traveled in each rank were calculated based on 2010 Road Traffic Census data.

(3) Sections where average travel speeds close to the representative value of each rank (8, 20, 35, 45, or 55 km/h) were extracted from the 2010 Road Traffic Census data.

(4) The motorcycles ownership rate in Japan is closest to Scenario 1. Therefore, the average travel speed extracted in iii) was used as the average travel speed in Scenario 1.

(5) Based on road traffic theory (Japan Road Association 1984) in Japan, simulations were conducted on the sections to calculate changes in travel speed with the conversion of road traffic mode from 4-wheel vehicles to motorcycles. However, it was supposed that for the rank 60 km/h and higher, no change in speeds would result from the conversion of the road traffic mode.

Table 4. Average travel speeds following the conversion of the road traffic mode

<table>
<thead>
<tr>
<th>Current travel speed class (km/h)</th>
<th>Scenario 1 motorcycles 9%</th>
<th>Scenario 2 motorcycles 20%</th>
<th>Scenario 3 motorcycles 50%</th>
<th>Scenario 4 motorcycles 80%</th>
<th>Scenario 5 motorcycles 100%</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 10</td>
<td>8.9</td>
<td>15.6</td>
<td>22.1</td>
<td>28.9</td>
<td>35.1</td>
</tr>
<tr>
<td>10 - 30</td>
<td>18.5</td>
<td>20.2</td>
<td>23.3</td>
<td>26.7</td>
<td>29.0</td>
</tr>
<tr>
<td>30 - 40</td>
<td>32.7</td>
<td>38.2</td>
<td>39.2</td>
<td>40.2</td>
<td>40.9</td>
</tr>
<tr>
<td>40 - 50</td>
<td>47.3</td>
<td>49.0</td>
<td>49.2</td>
<td>50.3</td>
<td>51.1</td>
</tr>
<tr>
<td>50 - 60</td>
<td>50.1</td>
<td>50.7</td>
<td>51.6</td>
<td>52.5</td>
<td>53.1</td>
</tr>
<tr>
<td>60 -</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

4.1.5 CO₂ emissions factors by vehicle type

Based on the relationship between 4-wheel vehicle travel speeds and CO₂ emissions (National Institute for Land and Infrastructure Management 2003), CO₂ emissions corresponding to average travel speeds for 4-wheel vehicles following conversion of the road traffic mode were first calculated. Subsequently, by taking the weighted average of CO₂ emissions factors based on the vehicle kilometers traveled ratio in the Road Traffic Census, CO₂ emissions factors for 4-wheel vehicles were obtained.

Regarding motorcycles, CO₂ emissions corresponding to average travel speeds were first calculated based on the relationship between motorcycle travel speeds and CO₂ emissions (Manabu DOHI et al. 2013). Subsequently, by taking the weighted average of CO₂ emissions factors based on the vehicle kilometers traveled ratio in the Road Traffic Census, CO₂ emissions factors for motorcycles were obtained.

4.2 Results of trial calculation

In Indonesia, the CO₂ reduction resulting from the conversion from 4-wheel vehicles to motorcycles was calculated to be approximately 21 million tons by using the baseline as the reference. Meanwhile, the CO₂ increase caused by the conversion from motorcycles to 4-wheel vehicles was calculated to be approximately 46 million tons. In Japan, the CO₂ reduction resulting from the conversion from 4-wheel vehicles to motorcycles was calculated to be approximately 66 million tons by using the baseline as the reference. Meanwhile, the CO₂ increase caused by the conversion from motorcycles to 4-wheel vehicles was calculated to be approximately 1 million tons. (see Figures 11).

Figure 11. Results of trial calculations
5. CONCLUSION

a. There is weak relationship between manufacture and CO₂ emissions. The manufacture 2 shows the lowering value of CO₂ emissions by 15%. Meanwhile there is weak relationship between engine displacement and CO₂ emissions. Engine displacement of 111-149 cc tends to reduce CO₂ emissions ranging from 17% to 43% compared with another engine displacement of ≤110 cc and ≥150 cc.

b. There is a weak correlation between service life and CO₂ emissions. Classification of motorcycle service life of one to six years, the CO₂ emissions are relatively the same. The service life has insignificant different values of CO₂, i.e. the highest value with the lowest only differs by 15%. The reason is because the technology in the last five years relatively the same using injection technology.

c. There is a quite strong relationship between travel speed and CO₂ emissions, the value on the travel speed of 10 km/h has the highest CO₂ emissions value. The travel speed of 60 km/h has optimum engine performance so that fuel consumption may be lower than low speed and high speed which is require greater engine performance.

d. In countries that have higher ownership rates of motorcycles compared with 4-wheel vehicles, it is expected that the conversion from motorcycles to 4-wheel vehicles lead to a dramatic increase in CO₂ emissions, whereas the CO₂ reduction effect resulting from the conversion from 4-wheel vehicles to motorcycles is small.

e. In countries that have higher ownership rates of 4-wheel vehicles compared with motorcycles, it is expected that the conversion from 4-wheel vehicles to motorcycles lead to a dramatic reduction in CO₂ emissions, although the increase in CO₂ emissions resulting from the conversion from motorcycles to 4-wheel vehicles is small.

f. Motorcycles are not suited to inter-city transportation such as logistics, though they are suited to relatively short trips such as commuting within a city. Therefore, it is necessary to examine for what kinds of trip the conversion from motorcycles to 4-wheel vehicles should be controlled. Because motorcycles pose a higher risk of traffic accidents than do 4-wheel vehicles, it is necessary to examine road safety measures and the measures will be able to contribute CO₂ reduction in the future.

REFERENCES
Indonesia Motorcycle Industry Association, (2012), Statistic of Motorcycle in Indonesia, Jakarta.
UNFCCC/CCNUCC, (2006) Indicative simplified baseline and monitoring methodologies for selected small-scale CDM project activity categories
National Institute for Land and Infrastructure Management. (2003). Calculation of emission factor of CO₂, NOx and SPM to use for the calculation of the quantitative index, Tsukuba.
Manabu DOHI., and Agah M. MULYADI.(2013), Bilateral joint research on Environmentally Friendly Transport System, using Motorcycle, 14th REAAA Conference, REAAA.
Minimizing Congestion on the Trans-Asian Highway

**PAPER TITLE**

(90 Characters Max)

Minimizing Congestion on the Trans-Asian Highway

**TRACK**

Integrated Mobility/Intelligent Transportation Systems/Sustainable Transport

**AUTHOR**

(Capitalize Family Name)

Paul MINETT

**POSITION**

Chairman

**ORGANIZATION**

Ridesharing Institute

**COUNTRY**

New Zealand

**CO-AUTHOR(S)**

(Capitalize Family Name)

**POSITION**


**ORGANIZATION**


**COUNTRY**


**E-MAIL**

paulminett@tripconvergence.co.nz

**KEYWORDS:**

Trans-Asian Highway; Congestion; Complexity Model; Commons Model; Ridesharing Institute;

**ABSTRACT:**

As surely as night follows day, highway completion is followed by traffic congestion. Designed to speed people to their destinations, the highways attract so much traffic that they defeat their own purpose. Taken across entire networks, traffic congestion imposes significant economic, social, health, energy, and environmental costs.

Transport authorities are applauded for their achievements on roads and transit, but existing levels of traffic congestion have arisen due to underinvestment in policy, funding, and enquiry into getting travellers to take a greater share of the responsibility for keeping traffic moving.

This submission explores the proposition that high levels of traffic congestion are not a transportation-engineering problem, and that instead the problem is behavioral and has to do with the way communities choose to use the road resource that is available to them. It questions whether the engineering thinking that has given rise to the situation is likely to resolve it, and suggests that new thinking is needed.

It suggests that looking through a different lens might provide some insights and alternative approaches to dealing with the challenge of traffic congestion, and explores the complexity model and the commons model as potential approaches for new thinking. It suggests that consideration be given to experimentation designed and led by an institution that is separate from the existing governance of the highway system.
Minimizing Congestion on the Trans-Asian Highway

Paul Minett¹

¹The Ridesharing Institute
Email for correspondence: paulminett@tripconvergence.co.nz

1 INTRODUCTION

The literature provides many methods for reducing traffic congestion. Some of these solutions have been found to have an impact, for some of the time. Some of them have had perverse impacts, seeming to increase the traffic they were intended to reduce. Even when the solutions work there appears to be no strong track record of persistent congestion reduction: before long the traffic grows again and more measures are needed. Even in Curitiba, Brazil, a city renowned for its successful bus rapid transit system, congestion continues to be a problem (Economist Intelligence Unit 2010).

Some transportation engineers scoff at the suggestion that congestion can be beaten, and some even characterize it as an indicator of success for an urban area. Travellers complain about it, economists work out how much it costs, politicians propose grandiose schemes that they claim will tame it, and yet it seems to inexorably grow along with the increasing urbanization of human life on the planet, and increasing levels of vehicle ownership.

Certainly in the short term, for congestion mitigation for major road projects, there is evidence of capability to reduce traffic flows through targeted strategies. But once the project is over the traffic returns to normal. The question is: why? Why can the flow of traffic not be managed so that there is persistently no congestion, or at least so that there is so little congestion that it does not need to be thought of as a ‘problem’? What should transportation engineers do? The objective of this paper is to explore possible responses.

Observing that the traditional engineering responses do not make a difference on the scale or with the persistence that is needed, this paper makes the unusual proposition that high levels of congestion are not actually a transportation-engineering problem. It suggests that the problem is behavioral, and that it has to do with the way communities choose to use the road resource that is available to them. The evidence is all around us, an inescapable conclusion once the problem is viewed through this different lens.

Given that the world turns to transportation engineers to solve congestion, what would be the implication of such a conclusion? Should the world turn to someone else instead? How would that work? Or should transportation engineers seek new insights that would help them find different answers? Does the problem arise from the way engineers think and see the world? Assuming that the current levels of congestion are the result of an accumulation of solutions designed by transportation engineers, can we really expect the thinking that has created the current situation to be able to resolve it? Is new thinking needed? If so, what? And what would be the relevance of this for efforts to minimize congestion on the Trans-Asian Highway?

The paper explores why transportation engineers have been the go-to guys for congestion relief for the past half-century, and the sorts of solutions they put forward. It checks in with some interesting types of congestion found on the Trans-Asian Highway. It explores the idea that new thinking is needed and suggests two models that might have relevance: the complexity model and the commons model. It discusses the potential for these new models to help find new methods to minimize congestion, and concludes with implications for policy and practice.

---

¹ The term ‘congestion’ will be used throughout the rest of the paper, meaning ‘traffic congestion’.
2 CONGESTION AND TRANSPORTATION ENGINEERS’ SOLUTIONS

Over the past few decades, as technologies have developed, costs of vehicles have fallen, access to individual mobility has increased, and populations have exploded, the highways\(^2\) have filled up. While motorization has made travel faster, the interactions between large numbers of vehicles traveling concurrently slowed it down, especially in peak periods. As long as the peak concurrent vehicle demand did not exceed the capacity of the highway, traffic slowed but total throughput increased. Once total concurrent vehicle demand exceeded the capacity of the highway, total hourly peak period throughput decreased, and the condition that most people experience as ‘congestion’ (stop and start traffic) became more and more common.

For a long time the expected solution to congestion was (and in some cases still is, see Figure 1) the expansion of the amount of highway. The logic was simple: once two lanes were full, add a third; when that filled up, add a fourth; and so on. Responsibility for the free flow of traffic rested with transportation engineers (they were the obvious go-to guys), and they stayed out of trouble by predicting the demand for highway capacity, and providing enough capacity to meet that demand. This situation persists today, except that in many places the highway right-of-way has been fully built out, the costs of expansion have become prohibitive and funding problematic, and sectors of society have demanded that highway expansion should stop – especially because it seems to attract more traffic, and also because of evidence that it does not have the congestion-reducing impacts that it promises, and because of the environmental impact of large numbers of stop-and-go vehicles.

Figure 1: A Sign Heralding Highway Expansion in Auckland, New Zealand

A second solution set, also in the transportation engineer’s toolbox as a natural extension of the first, was the expansion of public transportation. Most highway systems had some amount of public transportation to provide mobility for those who did not have access to a vehicle. Once again the logic was simple: addition of buses or trains would reduce the need for driving, and the demand for highway capacity could be balanced through the provision of buses, or even better through the provision of trains that ran on separate rights of way. To many, public transport was automatically a good thing and should be expanded as much as possible - ignoring the fact that vehicle ownership was exploding and that once they had a vehicle, people seemed to prefer to drive, especially compared with taking public buses. Increasing provision of buses might reduce the need for driving, but unless people preferred or chose not to drive, increasing buses did not reduce the demand for driving, and certainly not at levels that would avoid growth of congestion.

A third solution set, referred to variously as ‘travel demand management’, or ‘mobility management’, sought to reduce demand for peak period vehicle trips by encouraging people to use some alternative to driving, such as walking, cycling, carpooling, vanpooling, taking public transport, avoiding the

\(^2\) Except where the context suggests otherwise, the term ‘highway’ is used in this paper as a catch-all term to refer to roads, streets, highways, freeways, etc. Where the term ‘road’ is used, it is in the context of a ‘local road’, generally thought of as less than a highway.
trip, or traveling at a different time. A comparison of levels of public funding and effort dedicated to highway construction, public transport provision, and travel demand management shows that this third approach has been a decidedly poor cousin, receiving a mere fraction of available funds – for example as low as 2.5% in a recent plan for Auckland, New Zealand (ARTA 2008). Decisions about which alternative solutions to support depend upon transportation engineers expectations that they will succeed.

A fourth solution set has come to the fore in recent years, that of ‘tweaking the highway’ and ‘maximizing highway to vehicle to vehicle communication’, commonly referred to as ‘intelligent transportation systems’ that would enable existing capacity to be filled up more completely because vehicles could travel closer together, or drivers could be alerted to problems and divert around congestion, change their decision to travel, or choose to use a different mode. These solutions do not change underlying demand for driving.

Most of the literature about congestion treats it as an urban condition. Table 1 is a list of methods for reducing congestion that can be found in the literature (it is not offered as a complete list).

<table>
<thead>
<tr>
<th>Table 1. Methods for Reducing Congestion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Expand Highway Lanes</td>
</tr>
<tr>
<td>Dedicated Highway Resource to space-efficient modes: Bus lanes, HOV lanes, transit priority traffic control</td>
</tr>
<tr>
<td>Better coordinate traffic lights, and other flow enhancements such as shoulder running and variable speed limits</td>
</tr>
<tr>
<td>Introduce Congestion Pricing</td>
</tr>
<tr>
<td>Introduce expand use of Shuttles for first/last mile of journey</td>
</tr>
<tr>
<td>Introduce Driving Restrictions: restrict by license plate number, vehicle make, vehicle pollution performance, or vehicle occupancy, to limit vehicle numbers in target places on target days</td>
</tr>
<tr>
<td>Expand Public Transport – buses</td>
</tr>
<tr>
<td>Introduce/expand Commute Trip reduction/mobility management marketing programs</td>
</tr>
<tr>
<td>Introduce/expand Vanpooling</td>
</tr>
<tr>
<td>Rationalize bus routes to reduce bus-caused congestion</td>
</tr>
<tr>
<td>Reduce parking supply</td>
</tr>
</tbody>
</table>

Source: Author’s compilation

A big problem with all but a few of these solutions is that while they clear space on the highway, that space gets filled by other vehicles in a phenomenon referred to as ‘induced demand’ or ‘latent demand.’ Even when the political will exists to implement some of the more challenging options such as the London Cordon Charge, eventually the traffic builds back up and even stronger political will is required to keep increasing the charge ahead of the demand. Lack of political will can be cited as a key reason that many of the above solutions do not get implemented.

But the opposite of latent demand is also known to exist. ‘Disappearing traffic’ is a phenomenon that occurs when the amount of highway is arbitrarily reduced or closed, and the total increase in traffic flows on adjacent facilities is less than the reduced flow on the constrained (or closed) facility.

In a 2013 United Nations publication entitled Combatting Congestion, Litman proposes an implementation hierarchy for many of these methods, a prescription especially for the urban context, as shown in Figure 2. (Litman 2013)

However, even following this 2013 prescription that could be referred to as ‘best practice’ or ‘good practice’, there is an absence of examples of persistent success. High levels of congestion return. This would suggest that in and of themselves these solutions are not sufficient. Something more is clearly needed: but what?
VIII. OPTIMAL CONGESTION SOLUTIONS

This analysis indicates that optimal congestion reduction involves the following steps:

1. Improve alternative modes, including walking, cycling and public transit, and where appropriate, programs that support ridesharing, carsharing and telecommuting. Provide targeted improvements on congested urban corridors, such as more frequent transit services on congested roads, and commute trip reduction programs at major employment centers.

2. Manage roadways to favor space-efficient modes, such as bus lanes on urban arterials with more than 20 buses per hour during peak periods, transit-priority traffic control systems, and High Occupant Vehicle (HOV) lanes on urban highways.

3. If possible, apply congestion pricing (variable tolls or fees that are higher during congested periods), with prices set to reduce traffic volumes to optimal levels (typically level-of-service C or D).

4. Regardless of whether or not congestion pricing is applied, implement efficient transport pricing reforms to the degree that is politically feasible, including road tolls, parking pricing, fuel price increases, and distance-based insurance and registration fees. These reforms may be justified on various economic efficiency and social equity grounds.

5. Implement support programs such as commute trip reduction and mobility management programs wherever appropriate.

6. Only consider urban roadway expansions if, after all of the previous strategies are fully implemented, congestion problems are significant and congestion pricing would provide sufficient revenues to finance all associated costs, which tests users' willingness-to-pay for the additional capacity. For example, if a roadway expansion would have US$5 million annualized costs, it should be implemented only if peak-period tolls on that road will generate that much revenue. Off-peak tolls can be used to finance general roadway costs, such as maintenance and safety improvements, but not capacity expansion.

Figure 2: A prescription for reducing congestion

3 CONGESTION ON THE TRANS-ASIAN HIGHWAY

The Trans-Asian Highway is a transcontinental highway system to promote regional cooperation/trade. (Regmi 2011). There has long been a strong association between development of transportation systems and economic prosperity, and this project is an example of faith in that relationship.

In order to minimize congestion on the Trans-Asian Highway, it would be important to understand the nature of congestion that could occur, and to choose tools that are relevant to the situation, if they exist. The Trans-Asian Highway program is a set of design standards, signposting agreements, and upgrade projects that would create a highway system that runs east/west and north/south across the Asian continent, linking with European highways to the west, and enabling greater levels of trade between countries across the continent.

A central feature of the program is that it maximizes use of existing infrastructure rather than constructing new highways. The network incorporates major highways and local roads and everything in between, with more than a quarter being local roads, many of which might not have a sealed surface. (Regmi 2011).

If the Trans-Asian Highway program succeeds, there would be an increasing flow of goods and travellers along the defined routes, representing growing trade within the region and between the region and other parts of the world. A key point is that success of the Trans-Asian Highway would add traffic to the local roads that make up the network.

In many cases the local roads are likely already congested by local usage, at least by 2014 even if they were not in 1959 when the program was initiated. On the Trans-Asian Highway, congestion could occur for different reasons than in other parts of the world, for example as a result of differing cultural expectations and uses for local roads that might reduce their vehicle-carrying capacity: encroachment by vendors, pedestrians, processions, non-motorized vehicles, or livestock, for example; and physical impairments such as monsoon rains.
A poignant example of non-standard congestion is provided in the book:  *Saigon’s Edge: On the Margins of Ho Chi Minh City* (page 171) (Harms 2011)

“The Trans-Asia Highway project seeks to create an economic windfall by clearing congestion in and out of the city. But this very process produces its own contradiction, because economic booms fill the streets. In order to prevent the tendency for the rate of profit to fall, capitalist growth depends on the constant willingness of the consumer to go out and buy things. In the same way that the greatest public performance of vibrant capitalism might be seen at the crowded shopping mall, in the case of Tân Thới Nhất the vibrancy of the emerging market economy contributes to the performance of place at the roadside market. And paradoxically, as the Tân Thới Nhất market realizes itself as a place to buy and sell, it starts to cut off the very road that has contributed to its prosperity. In the mornings, when the pace of selling is most fervent, a semi-circle of buyers and sellers gathers by the entrance to the marketplace. Impromptu bicycle and motorbike parking lots extend onto the surface of the roadway, and the mass of people surges outward into the lanes of traffic, which slows and compresses into a thin line trying to squeeze through the bottleneck thus formed. Motorbikes snake through, and cars and trucks have to honk to clear a path. The market thrives because of its position along the highway, the selfsame highway it cuts off with its success. The market bites the road that feeds it.”

Which of the congestion-busting solutions in the literature would deal with this type of congestion?

By the same token, the Trans-Asia Highway’s easternmost edge is in Tokyo, Japan, and it therefore passes through modern cities that will likely have the types of congestion anticipated in the literature.

4 NEW APPROACHES TO THINKING ABOUT THE CONGESTION ISSUE

In the introduction it was suggested that a high level of traffic congestion is a behavioral problem rather than a transportation-engineering one. Is it possible that looking through a different lens could lead to new ideas about how to manage congestion?

The evolution of transportation knowledge has been based on a model of predict and provide, where the expert transportation engineer is asked to identify the best solution and then implement that solution. This approach relies for success on an assumption that there can be ‘best practice’ in transportation with reliable cause-and-effect relationships for problem solving, and that the community will respond in a predictable way. The engineer relies on a book of standards that is adhered to, come what may. As new problems are encountered, new solutions are developed and codified in the expectation that if a similar problem is encountered elsewhere, it can be solved by the application of the standard solution. Often the standard solutions deliver unexpected results. (Vanderbilt 2008).

The evolution of the approach to community involvement about how the transportation system should be developed and managed has been based on a model of decide and defend, where the expert transportation engineer identifies the best solution and then follows a hopefully unassailable process to win community support, including making best use of available mechanisms for bringing outside funding for the preferred solution. The community is often engaged in the decision process because such involvement is a requirement for access to local funding, and because the community wants a solution, but while the process achieves their buy-in they are not necessarily committed to using the solution.

The applicability of standard solutions to complex problems, and the potential for the community to manage its own resources as a way of avoiding over-use, are the subjects of relatively recent thinking that could have relevance for the management of congestion. They are referred to here as the complexity model and the commons model.

*The Complexity Model*

Recent developments in systems thinking have led to the contention that different degrees of complexity of a problem should lead to different approaches to resolving the problem (the complexity model). Suggested as a sense-making framework, systems can be seen as simple, complicated, complex, or
chaotic. The problem-solving approach to a simple problem would be different to that for a complex problem. (Snowden 2010).

A recent blog post on the World Bank website explains the complexity model and uses transportation analogies to help clarify the distinctions:

“Dave Snowden’s research describes problems or systems as either:

(i) **simple** - in which the relationship between cause and effect is obvious and we can generate best practice;
(ii) **complicated** – in which the relationship between cause and effect requires expert knowledge and good practice;
(iii) **complex** – in which the relationship between cause and effect can only be perceived in retrospect and we use emergent practice; and
(iv) **chaotic** – in which there is no relationship between cause and effect.

“Fortunately, few life situations correspond to chaotic systems (e.g. river rapids), and we know life is never truly simple (!) so most difficult situations we encounter are either complicated or complex. Building a bridge is complicated; managing traffic systems is complex. Planners and experts are very good at solving complicated problems where subject matter expertise is critical and methodical analysis leads to proper diagnosis and solutions.

“The problem is that we’re not very good at distinguishing between complicated and complex systems. Complex problems require very different methods to solve. For starters, you can’t replicate a solution to a complex problem. And any one answer is unlikely to have a sustained impact. Promoting efficiency can lead to disastrous consequences because underlying conditions change (think traffic) and getting really good at doing the wrong thing is a big risk.” (Walji 2013).

It seems reasonable to conclude that congestion has been treated to date as if it is a complicated problem. If in fact it were a complex problem, then the above explanation would shed some light on why standardised solutions to congestion consistently fail to solve the problem.

Snowden suggests that complex problems should be solved by getting lots of different people from lots of different backgrounds and designing ‘safe-fail’ experiments that can be amplified if they succeed, dampened if they do not. (Snowden 2010). This is quite different to the usual approach to solving congestion.

**The Commons Model**

In 2009 the Nobel Prize for Economic Sciences was awarded to Elinor Ostrom for her “groundbreaking research demonstrating that ordinary people are capable of creating rules and institutions that allow for the sustainable and equitable management of shared resources.” (Indiana University 2014).

Long before Hardin wrote his famous article ‘Tragedy of the Commons’, published in Science in 1968, it had been observed that common resources were subject to overuse. Rational (self-interested) consumption by each individual could be expected to result in irrational excess consumption by the whole of the community, thereby destroying the common resource. Policies to avoid destruction of the common resource involved either privatisation of the resource, or state control. Privatization generally results in unequal distribution of benefits, while state control incurs excess costs and is generally inefficient. These policy choices assume that overuse will be the result in every situation, and by implication that the individuals concerned cannot collectively establish usage, monitoring, and enforcement agreements and rules that would result in a better net outcome than the costs and inefficiencies of state control, or the unequal distribution of benefits flowing from privatization. (Ostrom 1990).

Ostrom explains that a problem that is central to community solutions (rather than state control or privatization) is the free rider – the one who gets the benefit of the collective action but does not contribute to the joint effort. Her research identified a number of examples where sustainable self-managed shared
resources existed, and had existed over long periods of time without becoming tragedies, and that dealt with the free rider problem. (Ostrom 1990).

It seems reasonable to think of the highway as a shared resource. While the over-use that causes congestion does not destroy the highway in the same way that a pasture might be destroyed through over-grazing, congestion destroys the daily effectiveness of the highway to deliver efficient mobility. Seemingly rational use by individuals results in irrational excess consumption (congestion) by the whole community that destroys the benefits for each individual and imposes costs on all users.

The point is that this is occurring in spite of the highway being governed through state control: a policy setting that is designed to avoid overuse. The question is whether a different governance structure that gave some control or responsibility to the users in a way consistent with Ostrom’s findings could achieve a better, and lasting outcome.

5 DISCUSSION

Congestion results from an excess of concurrent drivers, given the amount of highway. In a Western urban context the amount of highway is considered fixed. In the urban/rural Trans-Asian Highway context the amount of highway might be variable depending on circumstances. Provision of public transport and other solutions might reduce the ‘need’ for driving, but they do not automatically reduce the demand for driving. Solutions that remove vehicles from the highway create spaces that are then filled due to latent demand, but closing facilities causes traffic to disappear.

Success of the Trans-Asian Highway program could be measured by the amount of inter- and intra-region movement of goods and people. Much of this traffic will be ‘through traffic’ adding on to the existing level of traffic on local roads. Congestion will impede the flow of goods and people, and the success of the program increases the potential that congestion will occur. A key observation about the congestion that does occur is that it will be local. Congestion in one place will not automatically mean there will be congestion in another place. And given cultural expectations about the use of the roads for activities such as processions and markets, the reasons for congestion in one location might be quite different to the reasons in another. Strategies for minimizing congestion in one location might also be different than those for another. One size will not fit all.

The question is therefore raised about whether congestion is a complicated, or a complex problem. A feature of ‘complex’ is that it cannot be solved with standardized solutions. If congestion is indeed a complex problem, the approach to solving the problem (minimizing or reducing or eliminating congestion) should rely less on experts who bring standardized solutions and more on experimentation and development of emergent practices in each local context. Experience to date suggests that experts do not hold the answer to reducing congestion on a persistent basis, so such experimentation is probably necessary.

If we choose to experiment, what should be the dimensions of the experiment? Highway use is governed through state control, and as far as minimizing congestion is concerned it is difficult to characterize state control as a successful model. Is the highway a commons, and could it be governed more successfully using a different model that involves the community of users?

As a commons, the highway is different to other types of common pool resources that Ostrom’s work explores. A key difference is that highways are expandable, so upon finding the shared resource full, more highway could be constructed – not the case with grazing land, fish, or an aquifer. However, once the limits of highway expansion are reached (either physically or fiscally) the resource seems more similar. Another difference is that over-using the highway today doesn’t necessarily damage it for tomorrow – but it does damage its effectiveness for today in a very predictable way. Finally, use of fossil fuels on the highway impacts on another common pool resource – the air we breathe and the environment in which greenhouse gases accumulate.

What are the chances that a different governance framework could be found in which individuals modify their rational self-interested use of the highway in the interests of the greater common good of reduced congestion? Alternatively, what are the chances that by taking a community development approach rather than a transportation engineering approach and treating congestion as a complex rather than a
complicated problem, that emergent practices could reduce congestion? Could the emergence of significant amounts of mobile technology make it possible to manage the free rider problem often found in community solutions to shared resource use?

Could a mechanism be developed that would have the traffic disappearance impact of closing a facility, without closing the facility?

These are valid questions and there may be many more that should be addressed through experimentation. If successful the experiments would lead to progress in minimizing congestion on the highways of the world, including the Trans-Asian Highway. To enable such experiments to have the broadest possible scope, consideration should be given to having them designed and overseen by an institution that is separate to the existing governance of the highway system.

6 CONCLUSION AND IMPLICATIONS FOR POLICY AND PRACTICE

The lack of examples of persistent congestion reduction suggests that the range of methods currently available either do not work, are not being implemented in the right way for them to work, or are not in themselves sufficient. It is proposed that congestion reduction be treated as a complex problem and experiments carried out in local contexts to find emergent solutions. It is recommended that every aspect of transportation management be brought into the frame for experimentation – even the governance of the highway system – and that insights about governance of common pool resources be incorporated into the experimentation such that the communities that use the highway take on some responsibility for the effective use of the highway.

REFERENCES

Economist Intelligence Unit (2010). Latin American Green City Index: Assessing the environmental performance of Latin America’s major cities, Siemens AG, Munich Germany.
http://www.thecrystal.org/assets/download/Latin-American-Green-City-Index.pdf
A Comparative Analysis of Child-Friendly Transportation between Canada and Indonesia

<table>
<thead>
<tr>
<th>TRACK</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>AUTHOR (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>ARIEPHIN, Handiyana</td>
<td>Researcher</td>
<td>Institute of Road Engineering</td>
<td>Indonesia</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CO-AUTHOR(S) (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>NIEKERK, Femke</td>
<td>Lecturer</td>
<td>University of Groningen</td>
<td>Netherland</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>E-MAIL (for correspondence)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><a href="mailto:handiyanariephin@pusjatan.pu.go.id">handiyanariephin@pusjatan.pu.go.id</a>; <a href="mailto:f.niekerk@rug.nl">f.niekerk@rug.nl</a></td>
<td></td>
</tr>
</tbody>
</table>

KEYWORDS:
Congestion, motorcycles, accuracy, Traffic Counting

ABSTRACT:
Congestion that occurs in big cities in Indonesia, one of the reasons is the use of motorcycles as a primary means of transportation in Indonesia. This happens because the public transportation in Indonesia is not able to serve and satisfy all road users in Indonesia. To overcome congestion worsens, the government is trying to calculate the motorcycle road sections weighed in Indonesia. But the difficulties of the government in Indonesia, that to calculate the motorcycle is still done manually, while the motorcycle counter software that has been developed overseas companies can not accurately calculate the motorcycle due to the different driving characteristics in Indonesia which tend to be aggressive and do zigzag maneuvers. Pusjatan and NILIM have worked for 3 years running IPT especially in developing software to detect motorcycles in Indonesia. Joint research for 3 years running accuracy values obtained from the calculation using the software motorcycle IPT in terms of camera angles and the condition of the day.
A Comparative Analysis of Child-Friendly Transportation between Canada and Indonesia

Handiyana Ariephin1
Dr Femke Niekerk2

1Institute of Road Engineering, Ministry of Public Work, Indonesia
Email for correspondence: handiyana.ariephin@pusjatan.pu.go.id

2Faculty of Spatial Planning, University of Groningen, Netherland
Email for correspondence: f.niekerk@rug.nl

Abstract As a part of the community, children always become a vulnerable group that part of the victim from heavy traffic pollution. This group has also limited independent mobility that caused them injury or death by traffic accident. In Canada, road traffic crashes are the leading causes of injury death for children and also in Indonesia injury deaths from pedestrian group are women and children. Due to this reason, some developed countries accommodate children right and needs in urban transportation infrastructure planning. Therefore, regulations, policies and guidelines of children right were issued.
This paper promoted the implementation of Children Friendly Transportation Planning (CFTP) initiative in Indonesia by comparing the cases in Canada. The main objective was to give recommendation of legal and policy instrument reformation toward children-transport planning to Indonesian government. This research used descriptive and comparative analysis to observe a political will, a regulation, and children participation method. The result shows that the government has to have strong political will to promote children right in transportation. It is important to revise regulation of children right and involving them in transportation infrastructure provision. It is expected that implementation of CFTP in Indonesia will be improved.

Keywords Children - Child-friendly - Transportation - Participation - Regulation

1. INTRODUCTION

Children are a vulnerable part of the community that could be a “victim” of rapid urban development. The meaning of victim here is degradation of their quality of life including diseases caused by heavy traffic pollution (Peason, R. et al 2000), limited independent mobility of children and youth (Tranter, P., Doyle, J.1996) and also children injury or death caused by traffic accident (Canadian Institute of Child Health 2000). In Canada, road traffic crashes are the leading causes of injury death for children over the age of one year. In Indonesia, about 65% of injury deaths from pedestrian group are women and children. These conditions push some developed countries to accommodate children needs and right in urban transportation infrastructure planning. They issued regulations, policies and guidelines which are accommodating children rights and needs.

How about Indonesia’s policy and regulation concerning children right in general and in transportation especially? In 2002, Government issued a law related to children protection (number 23/2003). In this law (chapter 22) mentions that government is responsible to provide appropriate facilities and infrastructure for child protection. This law indicates that every infrastructure and facility development should consider children needs. In the transportation sector, there are two main laws that relate to transportation infrastructure and facilities provision, Law no 38/2004 regarding road and Law no 22/2009 regarding Road traffic and transportation. Under these laws, there are Government regulations that explain more detail of those laws. They are Government regulation no.34/2006 regarding road, Government regulation no43/1993 regarding Road infrastructure. And of course for implementation purpose, there are so many regulations on the level of a ministry decree. However, none of those laws and regulations mentions or regulates transportation infrastructure and facilities especially for children or stressing on providing firm legal instrument toward
children-friendly transportation infrastructure. Although in Law no 22 which issued in 2002 it has mentioned that transportation infrastructure provision should consider children need but in law no 34 regarding road which is issued in 2006 this issues was not considered.

There is lack to accommodate children need in laws that regulate transportation infrastructure and facilities and seems that among laws were not supporting each other. There is also a gap in process on how we can gather children participation for determining transportation policy. The question arises are: how to put children right and need in transportation policy and planning on the right legal instrument. How to include children’s rights and need in transportation planning?

2. BACKGROUND OF CFTP

Child-Friendly transportation Planning (CFTP) is a new approach and terminology. There is not many literature or planning documents that discuss or explore this terminology. Limitation of the theoretical literature subject to this terminology conveys difficulties for searching what the definition is. In practice, there are so many probabilities that this terminology has been used in several countries in different term. Based on The Centre for Sustainable Transportation research, sources of information on CFTP are based on two potential similar sources of CFTP. One source is design guidelines for children which is published by The Dutch institute for Design and another is Barntespektiv pa planeringen (Child’s perspective on planning) publishes by Nie Nilsson in Swedish. But there is no further information to explain the definition of CFTP in detail.

This concept has strong related concept with Child-Friendly city (CFC). The lead is the CFC initiative of the United Nations Children’s Fund (UNICEF), which is at the forefront of efforts to consider children needs and aspirations in an urban environment. CFC is responding to the global trend towards urbanization, to recognition that children constitute between 20-50 percent of populations, and to commitments made in respect of the United Nations Convention on the Rights of the Child.

The City summit in Istambul in 1996 highlighted that well-being of children in the city is the best indicator of how the healthy city is. A Child Friendly City is a people friendly city, encouraging the participation of citizen – young and old- in its services and its planning. Meanwhile, based on Building Child Friendly Cities document (Unicef,2000), there is nine characteristic for determining whether a city has put “children first” (UNICEF, 2000) shown in table 1

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Children's participation</td>
<td>promoting children's active involvement in issues that affect them; listening to their views and taking them into consideration in decision-making processes</td>
</tr>
<tr>
<td>A child friendly legal framework</td>
<td>ensuring legislation, regulatory frameworks and procedures which consistently promote and protect the rights of all children</td>
</tr>
<tr>
<td>A city-wide Children's Rights Strategy</td>
<td>developing a detailed, comprehensive strategy or agenda for building a Child Friendly City, based on the Convention</td>
</tr>
<tr>
<td>A Children's Rights Unit or coordinating mechanism</td>
<td>developing permanent structures in local government to ensure priority consideration of children's perspective</td>
</tr>
<tr>
<td>Child impact assessment and evaluation</td>
<td>ensuring that there is a systematic process to assess the impact of law, policy and practice on children - in advance, during and after implementation</td>
</tr>
<tr>
<td>A children's budget</td>
<td>ensuring adequate resource commitment and budget analysis for children</td>
</tr>
<tr>
<td>A regular State of the City's Children Report</td>
<td>ensuring sufficient monitoring and data collection on the state of children and their rights</td>
</tr>
<tr>
<td>Making children's rights</td>
<td>ensuring awareness of children's rights among adults</td>
</tr>
</tbody>
</table>
known and children

Independent advocacy for children supporting non-governmental organization and developing independent human right institutions-
children’s ombudspersons or commissioners for children-to promote children’s rights.


In Canada, Child-Friendly Transportation Planning terminology come arises from the need and requirement of such kind guideline of land-use and transport planning which giving more attention to children and youth while in Europe, it is much known as ‘children admitted’ principle. There are three conceptual contents of a programme of mobility based on the “children admitted” principle (Kids on the move, 2002). Firstly is decision of principle, second is framework measures and the last is action. To achieve goal for this program, ability to recognize reality problem through political commitment is first condition to be fulfilled. After political commitment has been achieved, legitimizations of actions and organization structuring have to be done. After these elements provided, next step is to make the program realize through actions.

3. METHODS OF CHILDREN PARTICIPATION IN PLANNING

There are benefits of including children in planning processes: ‘first, the personal and intellectual growth of the individual involved; second, the synergy of ideas created by organizing groups to educate themselves and to propel (to drive) them to turn their ideas into action; and, third, the creation of another area in which community development can take place’ (Checkoway, Pothukuchi, and Finn 1995).

There are four approaches to involve children in planning process as follows: scholarly, practice, educational and right based. Kimberly L. Knowles-Yanez, 2009). Firstly is Scholarly (Non-Right-Based) Approaches. This approach and land use practices are grouped together because of the following: they tend to lack a tight link to a practice outcome, are conducted by academic researchers in geography and planning, and do not make the right-based claimed of the last category this review. This scholarly approach is sometime overlaps with the educational approach (Doddridge 2000; Talen and Coffindaffer (1999)). Secondly is Practice Approaches. The practice approach to involving children in land use planning activities takes place in three sometimes overlapping realms: public agency, planning consultant, and not-for-profit (NFP). Thirdly is Educational Approaches. The educational approach does what education does best – allow children the freedom to dream big, with no fear of real damage being caused by their doings. In these approaches, children learn that there are constrains as well as great possibilities. And lastly is Right-Based Approaches. This approach explores children’s rights to participate in decisions that affect their lives. Two subcategories inside right-based approach are approach to use child development data and then draw recommendation for the policy and approach that learn on how children evaluate the place what they desire.

As a part of society, children have right to be involved in decision making process. With their huge population in the world, about one third of world population, their thought and action are important factor in determining our ‘collective future’. Their involvement and their perception become one of the important factors for shaping and forming regulation and legal aspect as part of Institutional system in transportation systems. Knowles-Yanez (2009) categorized 4 approaches in a matter children participation in planning process.

4. COMPARATIVE ANALYSIS AND DISCUSSION

This research used descriptive and comparative analysis to observe a political will, a regulation, and children participation method.

**Political will**

In order to elaborate political will of compared countries, this paper dismantles ‘political will’ terminology refer to components that are proposed by Lory (2002). The paper discuss current evidence regarding to 3 points, there are: a sufficient set of political actor, actors’ genuine intend to support a commonly perceived and public campaign.
Sufficient set of political actor

Actors who play key role in transportation sector decision are not only government who has an obligation to produce some kind regulations but also several bodies outside government. Miharja (2009) mentions that there are two main institutions that involve in transportation planning they are Public institution and Private institution. Public institution can be distinguished into 2 bodies: Government and House of representative while private institution consist of Expert, Private sector and nongovernmental organization (NGO)

Due to government and planning system in Canada, every government level has their own responsibility to manage several sectors. It means that federal government will put their effort to manage only to sectors which relate to national scope. National government will produce instruments which only regulate all matter on national level for instance transportation system serving between provinces. Provincial government and municipal government are also will concentrate on their own. They will engage in producing polices and plans for all matters within their administration. In Canada, Transportation Canada, one of central government agency, is a key actor to establish kind of rule regarding transportation system especially for national scope. But for smaller scope, provincial and municipal governments are important actor for planning matters. Provincial government issues Municipal act to control their municipal government and planning act.

According to CFTP practice, many initiatives have been arouse from lowest government level or municipal government. This level of government much more sensitive to the issue on local level and they have to make plan to overcome the problem or issue. In practice level, municipal level is the key actors to develop and to implement the CFTP initiative. For instance, the project to provide kind of guidelines which integrating child-friendly land use and transportation planning was funded by Ontario Government. Furthermore, the guidelines also proposed to revise Ontario’s planning act for including children need (propose to put the additional sentence in article that accommodate children interest). In Canada, where every province can produce their planning act, it has huge probability that the concept is not found in every province. It depends on how province government look one issues whether it is important or not. In this case, central government has responsibility to lift CFTP issue up to be considered as national issue

In Indonesia, on National level, Ministry of Public Work and Ministry of Transportation are the main actor for executing transportation system planning for national scope for instance national spatial plan, national road network, inter-province public transport system etc. These departments are also having responsibility to establish guidelines, standards norms for transportation infrastructure. These guidelines and standards are used for giving consistent service for whole transportation infrastructure and these standards also are used by lower actors in provincial and municipality for providing transportation infrastructure in their regions.

Initiative from central government still has influences even in decentralization era (Miharja, 2009) and has a strategic position to direct planning work. For instance, for establishing road master plan, municipal road master plan should refer to provincial road master plan and so on. In context CFTP implementation which is effective in local scope, Indonesia central government role apparent important for establishing regulation instrument and policy instrument (technical guidelines).

For those comparison, central government is still has great influence to implement CFTP, especially in Indonesia.

Actors’ genuine intend to support a commonly perceived, effective policy solution.

To find actors’ genuine intending to support the initiative, this paper explored what actors’ proposed through their policy statement or actions program that have been made.

Canadian Government shows serious attention for giving safer transportation system for children. To support the issue, Canadian government through Transportation Canada, issued guidelines, action plans and actions program to give guidance for local government to provide better access for children. Non-govern and professional organization support the initiative by also producing guidelines to be used for planner. Canada government commitment to support sustainable transportation was showed by providing funding approximately $ 437,500 per year over eight years for sustainable transportation project including Active & Safe Routes to School (ASRTS) program. Active & Safe Routes to School (ASRTS) is a community-based initiative program that promotes the use of active transportation for the daily trip to school. It is a growing movement that promotes and celebrates children’s active school travel in Canada.

In Indonesia, the pilot project that has been executed regarding to children safety matter is school safety zone (Zona Selamat Sekolah (ZoSS program)). The program has been running since 2006 and has been implemented in eleven municipalities in Java. This project was central government responsibility and
ministry of transportation through Directorate general of land transportation as executing agency. This project has aim to give safer road environment for children around their school and improve children accessibility to cross the road safely by giving speed reduction zone and pedestrian crossing or zebra cross. This program is a significant initiative related to CFTP implementation in Indonesia. It was difficult to find out how much budget had been allocated for this pilot project but this project show Indonesia government commitment for support child-friendly transportation.

Based on developed countries’ experiences, Canada has allocated large amount financial support for CFTP initiative implementation. Several actions program and guidelines have been carried out and established in those two countries. Non-governmental organizations were also involved actively to support this initiative. In Indonesia, government supporting for improving child-friendly transportation initiative is still not as much as intensive Canada. It is hardly to find Organization non governments that put their activity concern to children right in transportation seriously. However, there is positive sign from Indonesia government to CFTP initiative through ZoSS program. Financial support for this initiative is not as much as Canada according to different economic orientation and prioritization.

**Public Campaign**

How intensive public campaign has been carried out to promote such kind issues can be used to identify how much actors’ understanding of one issue in transportation planning sector. With intense promotion and publication of specific issues will shift old view to certain issues or form a new one not only among planning actors but also in society. Promotion and publication will put the issue on the center of government and public view.

In Canada, awareness to the issues has been aroused not only from government but also from other actors such as NGO. ‘Green Communities Active & Safe Routes to School is a comprehensive community-based initiative that concern about the increasingly urgent demand for safe and walk-distance neighborhoods. Active & Safe Routes to School promotes the use of active and efficient transportation for the daily trip to school, addressing health and traffic safety issues.’ These communities actively promote programs that support sustainable transportation particularly active and safe route to school. Active means children can make trip to school independently by walking or cycling. Promotion and public thought forming are delivered through various media such as post card and internet homepage. Henry Orsini’s post card is one example of publication using postcard media that express child opinion about fair-tariff for children. Henry Orsini, is a transit activist and with Better Environmentally Sound Transport compared transit fares in 13 Canadian cities. He created a postcard campaign urging Vancouver’s transit authority, Translink, to lower the children’s fare to 50¢. At that time Henry Orsini age was the 9-year old. This effort will change public view for putting children need in front as citizen and their voice need to be heard.

In Indonesia, public campaign and programs regarding to child-friendly and child-safety in transportation sound weak. However, there are signs that government start to promote more sustainable, environment and child-friendly transportation. Program car free day is one program which encourages citizen to use bicycle and public transportation for their journey. NGO participation for publicizing child-friendly transportation seems not very serious. Very limited governmental organizations that put their concern for child-friendly transportation or children safety were indicated.

Public campaign is an effective measure for forming a political will to support an initiative or idea. A frequent public campaign will shape public opinion for the importance of one program or initiative. Canada is country that is actively engage in children safety campaign and child-friendly transportation campaign. One of the reasons is because children safety has become important and main issues. Different condition is happened in Indonesia while government does not put attention seriously on children safety issues. The easiest indicator for this condition is difficult to find accurate data number children involved in traffic accident. Public campaign program to educate people about the importance of children safety and child-friendly transportation is still limited.

**Regulation and Planning instruments**

Planning instruments become important for planning activities as a base for making plans and it will be a reference for planning practice. Discovering planning instrument will lead us to know how far planning actors aware for some issues or how much the issues is important for them by putting them into a bundle of policy instruments. These instruments will force planning actors to follow all things that have been put in.

In Canada, general policy framework for National transportation in Canada is Transportation Act. Regarding to government and planning system in Canada, Every Provincial government can issue their own municipal
act, planning act and several directives which will oblige every municipal plan to refer to acts and directives. Provincial government also can issue the acts relate to transportation system, for instance Ontario Government, there are several act has been issued relate transportation particularly road transportation such as Highway act.

Regarding to CFTP issues, children need in transportation is not accommodated yet in particular acts. However, many programs and initiatives on local level has been promoted and run to protecting and servicing children need in transportation and there are many guidelines which were published by nonprofit organization or professional organization not only to accommodate children right (Children friendly and land use guidelines by Centre of sustainable transportation) and need in transportation but also to provide space for children in planning process including transportation sector (A kid’s guide in Building Great Communities).

In Indonesia, National government has obligation to issue Act as instruments for planning activities. According to spatial planning, Spatial planning act no 24/1992 has role to regulate all things relate to activity system and later is renewed by Spatial Planning act no 26/2007. To manage road development, Government of Indonesia has issued a new Road acts that is Road act no 38/2004. For managing traffic system, government has issued traffic and road transportation ac no 14/1992 and amended by later act that is no 22/2009. These 3 acts are main planning instrument related to transportation system. Among those 3 acts, no indication has been found that regulate things related to provide better and safer access to children, not specifically mentioned. Disable people are mention clearly in Road Act but not including children and young people within. However, Children Protection Act 2009 article 22, state that government has obligation and responsibility to provide appropriate infrastructure for children. It means that Indonesia has put more awareness for giving better infrastructure for children and no excuse for technical department and agency to ignore children need particularly in transportation services and infrastructures. Act is the highest hierarchy in legal framework and has a force every actor mentioned within to fulfill their obligation.

By knowing every government level’s role and responsibility to produce policies or regulations, we will be able to propose right recommendation to put children right in transportation sector in proper regulation framework, for instance to put children right in the highest level of regulation framework. It would be has a great impact if the initiative can be accommodate on the highest level of regulation (acts). However, we cannot make a generalization that acts is one and only one instrument that has a big impact. This is also interconnected with planning system culture of the country. In Canada, putting children right will give more impact if it is written on provincial Planning act. Transportation act that is issued by Canadian federal government is more likely to regulate transportation on national scope whether CFTP is a form sustainable transportation concept which will be effective if implemented on local scope or maximum on municipality level (see School Travel Plan)

Public/Children Involvement

Based on Canada experience, not many studies have been carried out to explore children involvement in planning process especially transportation planning process. However several projects relate to safe route to school used discussion and observation method to find out children travel pattern. One of the projects used high tech equipment (PDA) to track children travel pattern. One of professional association was also establish manual to involve children in building their community. This manual is dedicated to planner as guidance to gather children opinion. Other practice shows that children involvement to express their preferences about transportation service and environmental condition are revealed through various media such as web homepage and postcard. According to those practices, most children involvements in Canada are more likely adopting scholarly approach and/or educational (sometime overlaps, Yanez 2005) which observation and interviewing with the children is the most common method of these approach.

In Indonesia, children involvement in planning in Indonesia seems not developed yet. For instance, Technical guideline for developing School safety zone (Zona selamat sekolah/ZoSs) program does not put children involvement or children perception into account. All the method is standardize and become same in every spot. Children travel pattern data were gathered from interviewing with parent and children behavior for crossing road is based on observation without any interviewing with them to get more detail data about the impact of ZoSS initiative for them.

The development of children participation in planning process and decision making has developed in Canada. Culture in expressing the thought has been firmly formed in children culture. It is common and used to for children in Canada to express and to give their opinion. No doubt, if children in those countries will actively involve and give their opinion about appropriate transportation infrastructure provision for them. Cultural background of Indonesia is different from those two countries especially in education culture and
system. Indonesia children are not used to express their thought freely. This would be a challenge to find good method to involve them in decision making process.

Table 2. CFTP components comparison table

<table>
<thead>
<tr>
<th>Component</th>
<th>Canada</th>
<th>Indonesia</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Political will</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Actors involved</td>
<td>- Provincial and Municipal government have important role to initiate children-friendly</td>
<td>Recently, only one government agency who concern to promote a program related to children safety.</td>
</tr>
<tr>
<td>- actors’ role in the development of children-friendly transportation</td>
<td>- Government agency is key actor for issuing policies</td>
<td>- Not many NGO are concern about children safety in transportation</td>
</tr>
<tr>
<td>- NGO, such as Center for Sustainable Transportation (CST) and Canadian Institute of Planner (CIP) are actively promoting child-friendly matters.</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Actors’ understanding of child-safety problem and the importance of child-friendly transportation (including Public Campaign)</strong></td>
<td>High level of children safety promotion to create public will of the children safety importance</td>
<td>Partial, not all actors are aware to the issues and problem. Not every government institution care to child safety issue</td>
</tr>
<tr>
<td><strong>Planning Instrument</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Transportation Act on National level</td>
<td>- Planning act issued by National Government National : Spatial Plan Act, RTRWN)-</td>
<td>- Plans are bundled in act and regulation for every government level. Act for every transportation system on national level is available (Activity, traffic and network)</td>
</tr>
<tr>
<td>- Planning act and Municipal on Provincial level</td>
<td>- Road Act</td>
<td>- No regulation specifically related to children need in transportation.</td>
</tr>
<tr>
<td><strong>Legislation</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- On National Level, transportation act as an umbrella for National transportation system</td>
<td>- Plans are bundled in act and regulation for every government level. Act for every transportation system on national level is available (Activity, traffic and network)</td>
<td>- Children protection act obliges government to provide services and infrastructure for children</td>
</tr>
<tr>
<td>- Planning act is issued on Provincial or Regional level to control municipal governments and their planning activities</td>
<td>- No regulation specifically related to children need in transportation.</td>
<td></td>
</tr>
<tr>
<td><strong>Child-friendly transportation related guidelines</strong></td>
<td>Child- and Youth-Friendly Land Use and transport planning Guidelines</td>
<td>ZoSS Technical guidelines</td>
</tr>
<tr>
<td><strong>Public/Children Involvement</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Intensive program and effort to put children ahead in involving children in decision making</td>
<td>No evidence that children has been involved in policy making or studies that discuss about children participation in transport plan decision making</td>
<td></td>
</tr>
<tr>
<td>- Scholarly and educational approach, No formal institution identified for young or children participation</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 2 shows similarities, differences of components toward child-friendly transportation planning between two countries. Through comparing those component we can see ‘pot’ that we have to ‘filled’ clearly by learning from other counties that in advance for providing safer and better access to children.
By listing countries’ experiences and effort we can find their effort for realizing CFTP. The listing table shows how CFTP can be formed refer to those components. We can see that in countries where CFTP has been promoted and implemented well, the condition of component toward CFTP show positive tendency. We can see that political will component, including actors’ support; public campaign; actions program, and public/children involvement shows high, strong and intensive motivation. Comparing to Indonesia context, some components still have to be improved for supporting CFTP even the awareness to provide better access and safety for children has been showed through ZoSS pilot project. The Indonesia weakness to the process of CFTP is no specific transportation system regulation on high level (Act, Government regulation) for ensuring that children are protected and serviced by good transportation infrastructure. A way to go there is by forming political will and in the end regulator maker, Government and house of representative, will put their attention to accommodate children right in regulation products. To forming the political will, public campaign has an important role. It will form the paradigm of transportation actor, public and private, to support the initiative and endorse transportation regulator and decision maker to put children need in policy bundle. And based on Indonesia context, public campaign to promote the importance of CFTP is still weak.

5. CONCLUSION
The comparative analysis of this study shows that political will, which is represented through government commitment to implement CFTP, is still weak. It has been observed that only national government who concern to promote a program related to children safety and not all actors (provincial and local government) are aware to the issues and problem. Limited numbers of NGO are concern about children safety in transportation.

In laws and regulation, there is still gap which meant that current regulations are missed to promote CFTP. None of transportation system acts put children right in to account, but children protection act require government to provide appropriate services and infrastructure for them. This will be ‘a first step’ toward CFTP and will be a ‘base stone’ for establishing lower level legal instrument (Government regulation and Ministerial decree).

Children involvement in planning in Indonesia seems not developed yet. No evidence that children has been involved in policy making or studies that discuss about children participation in transport plan decision making. In Zoss case, children travel pattern data were gathered from interviewing with parent and children behavior for crossing road is based on observation without any interviewing with them to get more detail data.

There are some recommendations that could be options for putting CFTP in front. Firstly, political support and government commitment should be encouraged for introducing and implementing CFTP concept. Second, put children right in law or regulation related to transportation infrastructure provision including their right to be involved in planning stage. Third is promoting children right in transportation infrastructure including involving other actors such as NGO. One of Canada’s program or initiative that can be encourage to be implemented in Indonesia is to starting making community based initiative that promote children safety in Transportation similar to Active and Safe Routes to School (ASRTS).

Acknowledgments  Many thanks are due to Prof. Evvy Kartini for her suggestions and guiding to write this paper.

REFERENCES


Government of Republic Indonesia: Traffic and Road Transportation Act 14/1992
Government of Republic Indonesia: Traffic and Road Transportation Act 22/2009
Government of Republic Indonesia: Road Act 38/2004
Government of Republic Indonesia: Children Protection Act 23/2003
Government of Republic Indonesia: Human Right Act 39/1999
Government of Republic Indonesia: Spatial Planning Government Regulation 26/2008
Government of Republic Indonesia: Road Government Regulation 34/2006
Government of Republic Indonesia: Spatial Planning Government Regulation 26/2008


Kruger, J.S. and Chawla, L (2002), "We know something someone doesn’t know": children speak out on local condition in Johannesburg”. Environment and Urbanization 14, no.2 pp.85-96


Rahmah A. Transportasi ramah anak; http://anak.i2.co.id/beritabaru/berita.asp


Technical guidelines School safety Zones, Directorate of Land transportation safety, Department of Transportation

The data on traffic injuries and mortality of the Transport Canada Web site; http://www.tc.gc.ca/roadsafety/tp/tp13951/2001/page3.htm


Understanding sustainability and planning in the UK. http://www.espace-project.org/old/reading.htm (accessed 01/08/09)

### PAPER TITLE
(90 Characters Max)

| Pavement Distress Caused by Bitumen Hardening and Methods to Overcome. |

### TRACK

### AUTHOR
(Capitalize Family Name)

<table>
<thead>
<tr>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Engineer</td>
<td></td>
<td>Malaysia</td>
</tr>
</tbody>
</table>

### KEYWORDS:

Hardening, Bitumen, Ageing, Polymerization
Pavement Distress Caused by Bitumen Hardening and Methods to Overcome.

Tan Ho Inn

1University of Putra Malaysia, Serdang, Selangor, MALAYSIA
Email for correspondence: HITan@littrak.com.my

ABSTRACT:

1 INTRODUCTION

It has been well established that the rheological properties of the bitumen (binder) affect the pavement performance. Rate of hardening bitumen binder is an important factor that affects the durability of a bitumen concrete. As a rule of thumb, bitumen below penetration of 20 is at the end of its useful life. Loss of binder efficiency and brittleness prevent the material from containing the stresses imposed by traffic, leading to the development of cracks. This process is most obvious in the surface course where it is most exposed to sunlight (ultraviolet radiation) and air.

The factors affecting the age hardening of bitumen binder are:

- Oxidation – Reaction of oxygen with the asphalt cement.
- Volatization – Evaporation of some of the absorbed gases in the bitumen binder volatilization (loss of volatile oils)
- Polymerization – Combination of similar molecules to form larger molecules.
- Separation - is the removal of oily constituents, Separation is removal of oily constituents, resin or asphaltenes from the bitumen binder as caused by selective absorption of some porous aggregates.

Figure 1: Mechanism of Bitumen Ageing
Excessive hardening of the bitumen binder will cause the bitumen concrete to be too brittle and low-temperature cracking to occur. It may also cause the bitumen binder to partially lose its adhesion and cohesion, and subsequently it may cause raveling (progressive disintegration of pavement material and separation of aggregates from it) in the asphalt concrete.

The formula is used to evaluate relative aging of asphalt cements of different grades and/or different sources. The extent of age hardening can be quantified in terms of penetration or viscosity as follows:

\[ \% \text{ Ageing Index} = \frac{\text{Viscosity of Aged Asphalt}}{\text{Viscosity of Original Asphalt}} \times 100\% \]

2 PAVEMENT DISTRESSES ASSOCIATED WITH BITUMEN HARDENING

a) BLOCK CRACKING

The occurrence of block cracking usually indicates that the bitumen binder has hardened significantly.
b) RAVELING

Raveling is the wearing away of the pavement surface caused by the dislodgement of aggregate particles and loss of bitumen binder. Such damage may indicate that the bitumen binder has age hardened significantly.

![Figure 4: Ravelling on Pavement](image)

---

c) TRANSVERSE (THERMAL) CRACKING

Shrinkage of the pavement due to low temperatures or hardening of the bitumen binder.

![Figure 5: Transverse Cracking on Pavement](image)

The rate of bitumen hardening is dependent on bitumen character/composition, mixing temperature; air voids content, climatic conditions, air temperature and surface area. It usually increases with increased temperature, increased air voids content in the bituminous mix, increased service (air) and surface area.
3 CURTAILMENT OF BITUMEN HARDENING

a) PRODUCTION STAGES
The first significant hardening of bitumen takes place in the pugmill or drum mixer where heated aggregate is mixed with hot bitumen. During the short mixing time, the thin film of bitumen is exposed to air at temperatures which range from 135-163ºC.

1. One of the factors affecting the age hardening of bitumen binder is Separation. Separation is removal of oily constituents, resin or asphaltenes from the asphalt cement as caused by selective absorption of some porous aggregates. Selection of aggregates, which is non-porous or less porous aggregate may reduce the rate of bitumen hardening.

2. If the bituminous material is stored bulk as a liquid, it is important to reduce exposure to excessive heat wherever possible. If bitumen is exposed to heat and air, there is an increased chance of an oxidative reaction occurring and the bitumen hardening.

3. For Long term storage bitumen binder, the storage silo should be insulated, heated and airtight. Long term storage may cause problem drainage and oxidation of bitumen binder. Therefore, it is recommended that for long term storage, the temperature should be reduced to below 100ºC during heating, it is also recommended to circulate the material. A lack of circulation means the only movement of the bitumen is due to convection currents and to avoid entrainment of air during circulation.

4. The temperature of the aggregates from the dryer is primary factor that determines the temperature of the final mixture. If the aggregates are too hot, mixing causes the bitumen binder to oxidize excessively, producing premature hardening and if the aggregates are not hot enough, coating the aggregate is difficult.

5. During the mixing in the drum mixer, the location of liquid bitumen binder inlet should be far from flame burner. Moving the bitumen binder inlet closer to the flame burner the bitumen binder may be oxidized.

6. The most severe hardening of bitumen occurs during the mixing process. The temperature of the mixture during production should be closely monitored. The temperature must be just high enough to provide good coating on the aggregate and allow to satisfactory compaction. When the mixture is heated more than needed, additional oxidation and loss of volatiles occur.

7. Bitumen heating temperature should be in the range of 140 C to 160 C and when delivered to the pugmill as recommends that bitumen shall not be exposed during storage and mix production to temperatures of more than 350F (177 C).

8. The mixing time in pugmill mixer should be as short as possible to obtain uniform bitumen coating on all aggregates. Excess mixing time tends to degrade the aggregates and oxidize the bitumen binder. The dry mixing time of aggregate and filler in the pugmill should be minimal. Usually not more than 10 seconds. The wet mixing time of bitumen, aggregate and filler should be no longer than needed to uniformly mix the aggregate and coat the aggregate with bitumen. Usually about 30 to 35 seconds. Unnecessary long wet mixing time reduce plant output and promote additional age hardening of the mix.

9. The temperature of mix immediately after discharge from the pugmill may be as low as can be to get good coating and compaction to reduce binder hardening.

10. The mix must contain sufficient bitumen binder to ensure an adequate film thickness around the aggregate particles, thus minimizing bitumen binder hardening or aging during production and in service. And offset the effect of high voids if the asphalt film is too thin, air which enters the compacted pavement can more rapidly oxidize these thin film. Thicker bitumen binder films
around the aggregate particles harden at slow rate compared to thin film around aggregate. Increased bitumen film can significantly reduce the aging. The compacted mix should not have very high air voids (increased permeability) which accelerate the aging process. Some type of stabilizer such as fiber and bitumen modifiers can be used to prevent draindown of the bitumen binder during construction.

11. The use of additives and modifiers, bitumen binder undergo oxidative hardening during production, construction and in service. Antioxidant additives such as lead compounds, carbon and calcium salt are available to minimize oxidative hardening of bitumen binder.

b) CONSTRUCTION STAGES

1. During construction delivery of bituminous mixture haul distance should be minimized as much as possible.

2. For long hauls, the loads should be insulated with tarps and insulated beds. When the weather is cool or the haul time is long, bituminous mixture should have a protective covering to prevent excessive cooling and the formation of crust on the surface. It is important, that protective covers be securely tied down to prevent air from getting between the cover and bituminous mixture and to maintain the temperature. Therefore all covers should lap over the sides of the truck bed and be tightly tied down all along the sides and rear of the truck bed.

3. Good compaction improves structural strength and resilience of the pavement which is increases resistance to rutting and reduces moisture penetration and age hardening. This also reduces the volume of air in the asphalt mixture. A poorly compacted bituminous mixture pavement could result in reduced fatigue life, accelerated aging, rutting, raveling and moisture damage.

4. Weather (temperature and wind) primary factors affecting compaction. No paving in the rain. Paving work should only be carried out in dry weather. Rain will reduce mix temperatures and thus affect compaction. Cavities will also be formed once the trapped moisture eventually evaporates. High air voids allow water and air to penetrate into the structure and resulting oxidation, raveling and cracking.

5. The achievement of optimum compaction of thin asphalt mixes is dependent on compaction being carried out at the correct temperature. Placement of thin asphalt layers on cold pavements will result in insufficient compaction and an increase in air voids. This, in turn, will lead to a greater rate of oxidation and accelerated ageing of the bitumen wearing course. This problem needs to be addressed at mix design stage so that optimum life can be obtained from the pavement.

c) IN-SERVICE STAGES

1. When pavements begin showing signs of distress due to ageing binder it may be possible to carry out repair works that will prolong the pavement life.

2. When the pavement cracks, do the crack sealing or repair immediately. Cracks allow water, air and sun light to penetrate into the structure then will age the asphalt pavement. Advanced cases can be very costly to repair and can lead to formation of potholes or premature pavement failure

3. A rejuvenator might be able to postpone the need for surface treatment for a year or two as it rejuvenates the oxidized bitumen at the surface.
4 CONCLUSION AND RECOMMENDATIONS

Pavement is designed to cater for more loading with little room for any kinds pavement distress, of which may be quite detrimental to the road servisibility to the users. It is well understood that excessive hardening of the bitumen binder will cause the hot mix asphalt layer to manifest signs of longitudinal and crocodile cracks prematurely as it becomes too brittle (less than penetration of 20).

It is therefore imperative as highway construction practitioners that we ensure and control as much as possible of all bitumen hardening in pavement, whether the premix in process, under construction or in service. To ensure long life, one should use as soft bitumen as possible without reducing stability below the minimum required to prevent displacement under traffic loading.

5 REFERENCE


A Study on Media Exposure among Malaysian Road Users for Effective Communication on Speed Cameras Implementation

Yusof Ghani¹

¹Malaysian Institute of Road Safety Research
Email for correspondence: yusofghani@miros.gov.my

1 INTRODUCTION

The Automated Enforcement System (AES) is one of the interventions commonly practised worldwide to reduce the number of deaths and injuries on the roads. In some countries, it is known as ‘speed camera’ or ‘safety camera’, but regardless of what it is called, the cameras automatically capture drivers who violate traffic rules, especially speeding and red light running. In Malaysia, the AES is considered long overdue if the number of deaths and injuries are taken into account. The road death in Malaysia is alarming with close to 7,000 people killed each year since the last 10 years. Speeding and red-light running have been identified as two major factors for road deaths.

Although the AES has been implemented in many countries around the world, especially among developed nations, it is almost unheard of among Malaysians till 2012. Therefore, instilling awareness on the AES among members of the public is necessary to ensure the success of this intervention. The road users will only support this programme through the success of an advocacy programme that explains the safety benefits of its implementation. This study was carried out in 2012, three months prior to the implementation of the AES to determine public awareness level on AES that served as a baseline information. The study was also to determine the sources of information where the respondents received information on AES, if any, prior to any advertising campaign by the government.

Intensive campaign to educate road users on the AES is, therefore, necessary. Given the relatively high cost of mass media advertising, it is essential to know what elements make a road safety mass media campaign effective and how future campaigns might be made more effective. This requires a scientific-outcome based evaluation (Wundersitz, 2010). Therefore, MIROS plays a critical role in evaluating the campaign, and the success of the research conducted would be able to determine the success or failure of the on-going campaign, and hence, suggestions for continuous improvement.

The outcome of the research would be useful for the improvement of the advocacy programme, identifying the strength and weaknesses of the social marketing plan for AES advocacy, and therefore, making it cost effective. Delhomme (2009) suggested that a campaign is a purposeful attempt to inform, persuade, and motivate population (or sub-group of a population) to change its attitudes and/or behaviour to improve road safety, using organised communications involving specific media channels within a given time period.

2 LITERATURE REVIEW

The unmanned camera has been implemented in many countries in the world, especially developed nations. In Germany, for example, as of April 1996, there were 593 units of unmanned cameras in operation available (Glauz 1998). In Victoria, Australia, the system has begun as early as 1983 after two years of trial period. In Switzerland, the trials of automatic enforcement of red light running began in the 1970s. Currently there are 119 automatic (unmanned) enforcement units in operation (83 for speed only, and 36 for red light and speed).

Speeding and red light running have been identified as leading contributors to fatal accidents. Besides the conventional approach such as police summonses, the use of automated cameras would make enforcement
more effective. Therefore, the Malaysian government has decided to install more than 800 cameras in accident prone areas and at selected traffic light intersection throughout the country. Research evidenced that speed cameras are effective in reducing crashes. For example, a study by Rodier, et. al. (2007) indicated that automated speed camera could reduce speed by 2 to 15 per cent, and reduces crashes by 9 to 50 per cent.

As the government has decided to implement the automated enforcement system, the road users must be made aware of it. This is to avoid misunderstanding on the purpose of the AES implementation and eventually make them abide by the traffic rules. On the other hand, poor understanding on AES would become a setback and probably foil its success. Leidemann (2002) stressed on the importance of advocacy to spread the message on speed cameras. He cited that lack of public involvement in the development of the automated speed enforcement programme may have contributed to the public backlash that eventually led the Hawaiian legislature to shut the programme down. In Malaysia, the idea of having AES does not bode well with the opposition Members of Parliament and majority of road users. This is probably due to lack of understanding on the objective of AES. Of course, this has taken place due to the absence of advocacy activities.

According to Rodier (2007), the type and extent of public outreach necessary to build public support for automated speed enforcement varies. It can include traditional public education and outreach methods, such as public service announcements, press releases, and posters, at the very beginning of the programme. Other programmes use the internet and media to maintain a dialogue with citizens about the benefits of the programmes.

3 OBJECTIVES

Generally, this research aimed to look at the respondents’ awareness on the AES which was going to be implemented in Malaysia. The specific objectives of the study are as follows; to determine the level of awareness on AES implementation in Malaysia, to identify the source of information on AES among respondents, and to determine the effect of AES towards behaviour change prior to its implementation of in 2012. As the AES is yet to be implemented, this study was carried out in the absence of advocacy activities as well as the enforcement activities. The outcome of the study at this stage would be used to benchmark the outcome of the next stage of research in line with the stage of AES implementation when its operation begins.

4 METHODOLOGY

The survey was carried out in six different localities in Malaysia. Self-administered questionnaires were handed out to 1,194 respondents through purposive sampling. This was to ensure that the data collected will represent the population of every locality which made of three major ethnicities, age group and gender. Data collection was carried at the locations where the AES cameras will be installed for the trial period. Altogether, there would be 14 cameras installed in these areas, 8 for speeding and 4 for the traffic light. The locations of the data collection are listed in the Table 1 below:

<table>
<thead>
<tr>
<th>Traffic light</th>
<th>Speeding</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ipoh</td>
<td>Putrajaya</td>
</tr>
<tr>
<td>Alor Setar</td>
<td>Sungkai</td>
</tr>
<tr>
<td>Kuala Lumpur</td>
<td>Sungai Petani</td>
</tr>
</tbody>
</table>

The first part of questionnaire was designed to determine respondents’ media exposure pertaining to the AES cameras. The medium (such as television, radio, or newspaper) in which the exposure towards the AES
cameras are obtained were asked in this part. The second part was designed for self-reported behaviour on the level of awareness of the AES camera. This is measured through three stages of behaviour change; namely the cognitive, affective, and conative level. The demographic at the end of questionnaire asked about age, gender and ethnic background.

5 FINDINGS AND ANALYSIS

i. Demographic analysis

The data was analysed for gender distribution and revealed an almost equal distribution of male and female with 52.1% and 47.9%, respectively. As for age distribution, the highest percentage is made up of those between 26-35 years of age (32%), followed by 16-25 years of age (31.5%). The smallest group are those above 56, with only 6 per cent of total respondents. As for the ethnic distribution, 70.2% of total respondents are the Malays, followed by the Chinese (17.5%), and the Indians (10.8%). This distribution does not represent that of the national ethnic distribution, but is actually representative of the population for locations where the data was collected.

ii. Awareness on AES implementation

The respondents were asked whether they know the existence of the AES camera which the government had planned or already installed by the time of data collection. Respondents in Kuala Lumpur, Putrajaya and Alor Setar scored the highest with 72.36%, 69.79% and 65.5% respectively. This was due to the visibility of the cameras which have been installed along their regular routes. However, a lower score was recorded in Ipoh, Sungai Petani and Sungkai which installation works had yet to take place during the time of survey. Figure 1 explains the respondents’ awareness on the existence of the AES cameras in their respective area:

![Bar chart for awareness on AES cameras existence at different locations](image)

Figure 1: Awareness on the AES cameras existence at different locations

iii. Sources of information on AES

Among four different media that provide information on AES, the radio scored the highest with 47.1%, followed by newspaper 37.6%, television (36%) and the internet (30.7%). The absence of the TV advertisement at this period explains its lower score. Figure 2 shows the sources of information on the AES.

![Bar chart for sources of AES information through different medium](image)

Figure 2: Sources of AES information through different medium
The analysis zooms into the types of newspapers which provided or published information on AES. As shown in Figure 3, the Malay language newspapers Berita Harian and Harian Metro appeared to be the leading AES news provider with more than 25%, followed by Utusan Malaysia and Kosmo. Top English newspaper The Star only scored about 11%, while Sin Chew Daily has the highest score among the Chinese language newspapers with slightly more than 5 per cent.

![Figure 3: AES information obtained through newspapers](image)

The source for AES information through TV was analysed from the available data. As shown in Figure 4, the TV3 appeared to be the main source of information with (33.1%), followed by the government TV stations RTM1 (30.1%) and RTM2 (11.3%). Apparently, the satellite TV package Astro which has about 100 channels (such as Discovery, National Geographic and ESPN) hardly air any information on AES with only 3.5% respondent said that they obtained information from here.

![Figure 4: AES information obtained from television channels](image)

The AES information was also obtained through various radio channels. Figure 5 shows that government-owned Radio Nasional ranks the highest among all radio stations with more than 15%, followed by the Malay-language private radio station Era and Sinar. Among the English language station, Hot FM leads the pack with more than 10%. The English station was followed by Chinese language stations My FM and 988 FM, both scored less than 6 per cent. For the Tamil language radio stations, THR Raaga scored less than 4%. The Figure 5 below shows the score among the Malaysian radio stations:
While online ranked at the bottom when compared to other media, the data was analysed to see the rank among different online media. Figure 6 shows that Facebook tops the list with 21%, followed by official government website (18%), online newspapers (14%) and blogs (10%). There was little mention about the AES in online radio or Twitter.

Apart from the traditional media and the internet, the study also looked into other sources which respondents obtained their information on AES. The word of mouth, AES signage, billboards, minister or VIP speeches were among the top sources of information where ‘other sources’ are concerned. Figure 7 shows the other sources of information where the respondents’ obtained information on AES.
iv. The Stages of Behaviour Change

Next, the analysis was done on the hierarchy of effect on the stages of behaviour change to determine respondents’ readiness to change their behaviour upon acquiring knowledge on AES. This was based on a set of questions that were grouped for different levels (cognitive, affective and conative). The average score was calculated for each group. As for the cognitive level, which is their knowledge on AES, the mean score was 3.84 out of 10. This shows that they had very low knowledge on the benefits of AES which was to be introduced. As for the affective variable, which described the respondents’ intention to change behaviour if the AES were introduced, the mean score was 7.33 out of 10, indicating that they were willing to change their behaviour should AES come to effect. Finally, for the conative variable, the mean score was 7.28 out of 10, which is high. This reflected that they will definitely change their behaviour once AES operation started, probably out of fear of getting summons. Refer to figure 8:

![Figure 8: Average mean on level of awareness on AES](image)

v. Sources of information on AES against the ethnic group

A cross tabulation analysis was carried out to determine the type of media that mostly served as source of information by respondents’ of particular ethnicities. The Malays mostly learned about the AES through word of mouth and radio; the Indians learned from radio and newspapers; the Chinese learned from radio and word of mouth; and, other ethnicities in the Malaysia learned from radio and word of mouth as well. This shows that word of mouth were fast spreading after the news appeared in radio or newspapers. In contrary to the norm, television was not a prominent source of information concerning to AES, at least during the baseline study phase. Figure 9 shows the sources of AES information against ethnicities:

![Figure 9: Cross tabulation on sources of AES information against ethnic groups](image)
vi. **The importance of information on AES**

Next, the data was analysed to determine the importance of the AES information among respondents. They were asked on the agreement from scale 1 to 10, of which 10 being the highest score. The respondents’ score were calculated for the mean score, and the outcome is shown in table 2 below:

<table>
<thead>
<tr>
<th>Statements</th>
<th>Mean Score</th>
</tr>
</thead>
<tbody>
<tr>
<td>Information on AES will make me a better driver</td>
<td>7.30</td>
</tr>
<tr>
<td>I will become a better driver once I understood about AES</td>
<td>7.54</td>
</tr>
<tr>
<td>I will continue looking for information on AES</td>
<td>6.83</td>
</tr>
<tr>
<td>I make efforts to look for information on AES</td>
<td>5.93</td>
</tr>
<tr>
<td>I often share information on AES with other drivers</td>
<td>6.50</td>
</tr>
</tbody>
</table>

Generally, the score indicated that respondents believe the information on AES will make them better drivers, particularly regarding speed limit and red light running. In addition, the respondents were interested with information on AES, and those who have information will also tend to share with others.

6 CONCLUSIONS

The study has shown that at this stage, prior to AES implementation whereby the advocacy and activities are absence, there was generally a low awareness on it. However, in places where the AES cameras have been installed but yet to be in operation, such as Kuala Lumpur, Putrajaya, Alor Setar and Sungkai, word about the AES were fast spreading.

In addition, there were also some media coverage, though generally low. The study indicated that at this stage, radio and newspapers gave the most information on the AES. In terms of media exposure where the respondents received the information, it was clear that respondents of different ethnic group preferred the media of their mother tongue.

Although there were low degrees of awareness on the implementation, the respondents’ knowledge on the benefits of AES remained low. This information would be useful to help the social marketing plan to advocate the benefits and the needs of AES for safer roads.

With regards to the hierarchy of change on road users’ behaviour towards AES, the study concluded that respondents are willing to accept AES and change their behaviour towards speeding and red-light running accordingly if they are knowledgeable about it. This was also due to the threat of summonses that comes with the implementation of AES.

The respondents also believe that the implementation of unmanned cameras would change them into better drivers and that the AES is a subject of interest which they wanted to be educated about. Generally, this study managed to gauge the level of road users’ knowledge towards AES. Therefore, it is recommended that communication campaign efforts to win public acceptance on the AES must be planned carefully based on the available evidence from this research.

To deal with this, the AES advocacy committee must properly plan and execute the social marketing campaign to sell the benefits of AES. In dealing with intricate society that consists of different ethnics and mother tongue, the social marketing campaign must utilise various mediums such as television, radio and newspapers and also billboards, banners, buntings and leaflets, to name a few. In addition, social media such as Facebook, Twitter and YouTube could also be used extensively, and made viral for greater reach among intended audience. This requires a delicate planning in choosing the media that best fit the targeted audience from the respective demographic groups to ensure optimum reach and understanding of the messages.

Rodier (2007) suggested that the type and extent of public outreach necessary to build public support for automated speed enforcement varies. It can include traditional public education and outreach methods, such as public service announcements, press releases, and posters at the very beginning of the programme. Other
programmes capitalise on the internet and media to maintain a dialogue with citizens about the benefits of the programmes.

As this study may be general in nature, it managed to pave way for some ideas on the exposure to the media among different ethnic groups in Malaysia, which largely speak the language of their own mother tongue. The choice of different media for different ethnic group requires the communication campaign on the AES to be planned accordingly given the budget constraint for the campaign.

Perhaps a detail evaluation on the specific programmes preferred by the target audience, particularly on television and radio, must also be researched to ensure that the optimum return on the money spent for the campaigns. In addition to the costlier traditional media such as television, radio and newspaper, the use of social media must also be explored in the most creative manner as for the messages on AES to penetrate the intended audience of information.

The evidence provided by this research certainly is useful in guiding the campaign planners to choose the right medium of communication for different target audience. In a multi-ethnic society in Malaysia, there is no ‘one-size-fits-all’ choice of media where message channelling is concerned. Thus, a study on media exposure proves valuable in communicating the messages AES in order to win support from every road users, which would eventually make Malaysian roads safer.

Similar study could be applicable in other countries which have diverse social and cultural backgrounds, in which different languages are spoken. While the messages could be the same across languages, the different channels of communication must be carefully determined to ensure an effective flow of communication. No advocacy programme would be successful without a good understanding on the audience, the messages, and the channels of communication in which the messages were delivered.

REFERENCES


PAPER TITLE
(90 Characters Max)
The Development of Motorcycle Crashes Prediction Model on Collector Roads By Using Generalized Linear Models

TRACK

AUTHOR
(Capitalize Family Name)
Machsus

POSITION
Lecturer

ORGANIZATION
The Diploma program of Civil Engineering, Institut Teknologi Sepuluh Nopember

COUNTRY
Surabaya, Indonesia

CO-AUTHOR(S)
(Capitalize Family Name)
Harnen Sulistio, Achmad Wicaksono, Ludfi Djakfar

POSITION
Professor & Associate Professor

ORGANIZATION
Civil Engineering Department, University of Brawijaya,

COUNTRY
Malang, Indonesia

E-MAIL
(for correspondence)
machsus@ce.its.ac.id ; machsusfawzi@yahoo.com

KEYWORDS:
Motorcycle accidents, generalized linear models, prediction model, collector roads, traffic flow, length of road, access points, traffic speed

ABSTRACT:
The phenomenon of traffic accidents in Indonesia is characterized by the high frequency of traffic accidents and the involvement of motorcycle. The involvement of this type of two-wheeled vehicles is the highest compared to other vehicle types. In addition to arterial roads, the frequency of traffic accidents in urban areas also often occurs on collector roads. This paper will be present the development of prediction model of motorcycle accident on collector roads in Surabaya, Indonesia. This study uses generalized linear model with a Poisson distribution and logarithmic link function approach, as well as the R software applications in the data processing. The location taken as a case study is urban roads. In this study, 69 sections of 120 collector roads in Surabaya are chosen. The result of this study indicates that the traffic volume, length of roads, number of access points, and the speed of traffic are significantly influential in describing a motorcycle accident on collector roads. These findings are expected to help the stakeholders in the field of traffic engineering to overcome the problems of traffic accidents on collector roads in Surabaya and other areas.
The Development of Motorcycle Crashes Prediction Model on Collector Roads By Using Generalized Linear Models

Machsus¹, Harnen Sulistio², Achmad Wicaksono³, Ludfi Djakfar⁴

¹²³The Diploma program of Civil Engineering, Institut Teknologi Sepuluh Nopember, Surabaya, INDONESIA
²³⁴The Department of Civil Engineering, University of Brawijaya, Malang, East Java, INDONESIA

Email for correspondence: machsus@ce.its.ac.id; machsusfawzi@yahoo.com

1. INTRODUCTION

    Portrait of road traffic accidents in Indonesia can be seen from the frequency of accidents data, the vehicles involved the number of victims and the material loss. The number of traffic accidents reached 109,776 in 2011, with 239,257 units of vehicles involved and a total of 176,763 victims. Moderate material losses were estimated at Rp 86.09 billion (Directorate General of Land Transportation, 2012). Portrait of road traffic accidents that occur in the city of Surabaya also reflects a phenomenon in Indonesia. Widyawastuti (2005) stated that the number of victims of road traffic accidents caused by motor-cycle is higher than other types of vehicles. Motorcycle accident is reported to have the highest proportion compared to other vehicle types, such as in Table 1 below.

<table>
<thead>
<tr>
<th>Types of Vehicles</th>
<th>units</th>
<th>2007</th>
<th>2008</th>
<th>2009</th>
<th>2010</th>
<th>2011</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Car</td>
<td>unit</td>
<td>236</td>
<td>174</td>
<td>268</td>
<td>172</td>
<td>272</td>
<td>224</td>
</tr>
<tr>
<td>Truck</td>
<td>unit</td>
<td>160</td>
<td>159</td>
<td>175</td>
<td>148</td>
<td>245</td>
<td>177</td>
</tr>
<tr>
<td>Bus</td>
<td>unit</td>
<td>17</td>
<td>10</td>
<td>18</td>
<td>17</td>
<td>16</td>
<td>16</td>
</tr>
<tr>
<td>Motorcycle</td>
<td>unit</td>
<td>901</td>
<td>776</td>
<td>978</td>
<td>883</td>
<td>1,487</td>
<td>1,005</td>
</tr>
<tr>
<td>Overall</td>
<td>unit</td>
<td>1,314</td>
<td>1,119</td>
<td>1,439</td>
<td>1,220</td>
<td>2,020</td>
<td>1,422</td>
</tr>
</tbody>
</table>

Table 1: Motor vehicle traffic accidents in Surabaya

Tabel 1 shows that from 2007 to 2011, the involvement in various types of motor vehicle accidents in the city of Surabaya on average, as follow: for the type of motorcycle, it was 70.7% (1,005 units), followed by automobiles of 15.8% (224 units), container vehicles amounted to 12.5% (177 units), buses by 1.1% (16 units). Although the number is fluctuates, the accidents involving motorcycles in Surabaya from 2007 to 2011 tended to increase with an average growth rate of 17.71% per year.

Furthermore, the portrait of a traffic accident on the highway can be analyzed from several parameters, including: the frequency of incident, the vehicles involved, the number of casualties (death, serious injury, and minor injuries), and the amount of material loss. It also can be seen from the comparison of the level of traffic accidents in the city of Surabaya against accidents that occur in the region of East Java province and nationally in Indonesia, as can be seen in Table 2 below.

Table 2. shows that the rate of traffic accidents in the city of Surabaya is higher than the average rates of the district or city, both nationally and locally in East Java. The high rate of accidents in the city of Surabaya occur in all indicators which include: the frequency of events (3.1 events / day), the vehicles involved (5.5 units / day), the number of victims (1 person dies / day) and loss of material (0.85 billion / year). Similarly, traffic accidents that occur in the province of East Java is also higher than the provincial average rates as well as provincial average rates, nationally.

The following is the comparison rate of traffic accidents in the city of Surabaya on the district/city average rates nationally. The indicator of event frequency shows that traffic accidents in Surabaya is 5.2 times greater than the average value of districts/city average rates nationally. Meanwhile, the indicator of vehicles involved in traffic accidents shows that it is 4.3 times greater than the national average rates. Similarly, the indicator of the number of casualties in traffic accidents is 5.7 times greater than the national average rates, while the indicator material loss of traffic accidents is 5.0 times greater than the national average rates.
Table 2. Comparison of rate of traffic accidents in the city of Surabaya, East Java and Indonesia in 2011

<table>
<thead>
<tr>
<th>Description</th>
<th>Unit</th>
<th>Indonesia National 1</th>
<th>Provincial Ave.</th>
<th>East Java 2</th>
<th>District Average</th>
<th>Surabaya City 3</th>
<th>Accident Rate, Surabaya to :</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>National</td>
<td>Provincial</td>
<td>East Java</td>
<td>National</td>
<td>Provincial</td>
<td>Surabaya City</td>
</tr>
<tr>
<td>Frequency of incident</td>
<td>incident/year</td>
<td>109.776</td>
<td>2.889</td>
<td>21.989</td>
<td>216.5</td>
<td>578.7</td>
<td>1.119.0</td>
</tr>
<tr>
<td></td>
<td>incident/day</td>
<td>301</td>
<td>8</td>
<td>60</td>
<td>0.6</td>
<td>1.6</td>
<td>3.1</td>
</tr>
<tr>
<td>Vehicles involved</td>
<td>unit/year</td>
<td>239.257</td>
<td>6.296</td>
<td>37.208</td>
<td>471.9</td>
<td>979.2</td>
<td>2.020.0</td>
</tr>
<tr>
<td></td>
<td>unit/day</td>
<td>655</td>
<td>17</td>
<td>102</td>
<td>1.3</td>
<td>2.7</td>
<td>5.5</td>
</tr>
<tr>
<td>Fatalities</td>
<td>person/year</td>
<td>31.185</td>
<td>821</td>
<td>5.499</td>
<td>61.5</td>
<td>144.7</td>
<td>361.0</td>
</tr>
<tr>
<td></td>
<td>person/day</td>
<td>85</td>
<td>2</td>
<td>15</td>
<td>0.2</td>
<td>0.4</td>
<td>1.0</td>
</tr>
<tr>
<td>Serious injuries</td>
<td>person/year</td>
<td>36.767</td>
<td>968</td>
<td>3.925</td>
<td>72.5</td>
<td>103.3</td>
<td>580.0</td>
</tr>
<tr>
<td></td>
<td>person/day</td>
<td>101</td>
<td>3</td>
<td>11</td>
<td>0.2</td>
<td>0.3</td>
<td>1.6</td>
</tr>
<tr>
<td>Slight casualties</td>
<td>person/year</td>
<td>108.811</td>
<td>2.863</td>
<td>24.979</td>
<td>214.6</td>
<td>657.3</td>
<td>680.0</td>
</tr>
<tr>
<td></td>
<td>person/day</td>
<td>298</td>
<td>8</td>
<td>68</td>
<td>0.6</td>
<td>1.8</td>
<td>1.9</td>
</tr>
<tr>
<td>Loss of material</td>
<td>Billion Rp.</td>
<td>86.09</td>
<td>2.27</td>
<td>25.10</td>
<td>0.17</td>
<td>0.66</td>
<td>0.85</td>
</tr>
</tbody>
</table>

Description: Average calculation based on the assumption nationally with 34 provinces, and 507 districts of the city, as well as 38 cities in the provinsional district of east java

Source: 1 PPDA, Directorate General of Land Transportation (2012)
2 Traffic Accident Unit, East Java Regional, Indonesian National Police
3 Traffic Accident Unit, Surabaya Police, Indonesian National Police

Further discussion is about the results of the comparison rate of traffic accidents in the city of Surabaya on the average rates of the district/city in East Java region. The frequency indicators show that the incidence of traffic accidents in Surabaya is 1.9 times greater than the average rates of the district/city in East Java. The indicators of vehicles involved is 2.1 times greater than the average rates of East Java. Worse than the previous indicators, the indicator of the number of casualties in Surabaya is 3.0 times greater than the average rates of East Java. While the indicator material loss is 1.3 times greater than the average rates East Java.

To estimate the frequency of a motorcycle accident on roads, accident prediction models need to be made. Accident prediction models can also be used to identify and determine the relationship among the factors that influence, such as geometric, environmental and operational factors (Natalya et al, 2013; Keshava et al, 2013). In the establishment of motorcycle accident modelling the relationships among the variables that are used need to be considered (Nambuusi et.al. 2008). The output of accident prediction model development is expected to be used in the planning and the implementation of action programs on road safety (Olescenko and Dobromirov, 2013; Davati and Arani, 2012).

Generalized linear models used in the modeling, because the traffic accident data is no longer assumed to be normally distributed. Poisson distribution and the negative binomial distribution are kinds of distribution that is often used in the previous research. In this study, Poisson distribution or negative binomial and logarithmic link function are also used. This is because they are able to describe the distribution of randomly, discretely, and non-negatively, which definitely represent the characteristics of the traffic accidents (McCullagh and Nelder, 1989; Harnem et al, 2006).

2. MATERIAL AND METHODS

The locations of this research are on collector roads in Surabaya, Indonesia. Data collected includes: number of motorcycle accidents, traffic volume, length of roads, number of lanes, number of access points (openings), width of the road, road median, traffic speed, number of directions, and road shoulders. The roads selected are based on the condition between 2009 and 2012 with following criteria: a. only on collector roads; b. no significant change in characteristics over the study period; c. roads that do not have frontage road; and d. geometric characteristics of each road segment is relatively the same or relatively homogeneous. Based on those criteria, this study selected 69 collector roads in the city of Surabaya.

The motorcycle accident data was obtained from the Traffic Accident Unit at Police Regional Directorate of Traffic East Java and Surabaya Police of Indonesian National Police, from 2009 to 2012. Traffic volume data; the speed of the vehicle; and collector roads geometric acquired by Surabaya City Government Agencies, include: the Transportation Department, Department of Highways, Urban planning and development Office.
The response variable or dependent variable used in this study is the number of motorcycle accidents per year. The explanatory variables include: traffic volume, road length, number of lanes, number of access points, the width of the road, road median availability, traffic speed, the number of direction and road shoulders (Machsus, et al. 2013).

3. RESULTS AND DISCUSSION

In this study, this prediction model building is divided into two stages; univariate analysis phase and multivariate analysis phase. For statistical data processing, R Software, Windows i386 - 32bit version 3.0.3, (2014) is used. Univariate analysis phase is obtained that any explanatory variables that are used to qualify the significance (<0.05), as shown in Table 3 below. This means that each of the explanatory variables is influential to the motorcycle accident when a partial analysis is conducted. Many studies have shown that the tpr. parameter estimates significant at the 95% confidence level, or significance of <0.05 [9, 10]. If there are any parameters of the explanatory variables are not eligible in the significance, then it is removed and will not be involved anymore in the next analysis phase, such as variable LN, RM and SHDW.

Table 3. Univariate Analysis of The Terms

| Parameter | Estimate | Std. Errors | z value | Pr(>|z|) | Significant at 0.05 |
|-----------|----------|-------------|---------|---------|---------------------|
| Intercept | -12.7638 | 1.2146      | -10.51  | <2e-16  | Yes                 |
| FLOW      | 1.7656   | 0.1515      | 11.65   | <2e-16  | Yes                 |
| Intercept | -8.81842 | 0.66380     | -13.29  | <2e-16  | Yes                 |
| LR        | 1.37222  | 0.08742     | 15.70   | <2e-16  | Yes                 |
| Intercept | 1.06470  | 0.20382     | 5.224   | 1.75e-07| Yes                 |
| LN        | -0.02682 | 0.06770     | -0.396  | 0.692   | No                  |
| Intercept | 1.243605 | 0.130081    | 9.56    | <2e-16  | Yes                 |
| AP        | -0.020068| 0.009039    | -2.22   | 0.0264  | Yes                 |
| Intercept | 1.37164  | 0.19016     | 7.213   | 5.47e-13| Yes                 |
| RW        | -0.03257 | 0.01543     | -2.111  | 0.0348  | Yes                 |
| Intercept | 0.97495  | 0.08865     | 10.998  | <2e-16  | Yes                 |
| RM        | 0.04527  | 0.15820     | 0.286   | 0.775   | No                  |
| Intercept | -4.54438 | 0.379743    | -11.97  | <2e-16  | Yes                 |
| SPEED     | 0.138799 | 0.00847     | 16.38   | <2e-16  | Yes                 |
| Intercept | -1.4788  | 0.5447      | -2.715  | 0.00663 | Yes                 |
| CN        | 1.3273   | 0.2803      | 4.736   | 2.18e-06| Yes                 |
| Intercept | -0.0612  | 0.17120     | -0.358  | 0.72    | No                  |
| SHDW      | 0.76025  | 0.09619     | 7.904   | 2.7e-15 | Yes                 |

The influence of the explanatory variables simultaneously to the model of accident can be identified by multivariate analysis. In multivariate analysis, it is found that only a few explanatory variables are eligible in the significance (> 0.05), as shown in Table 4 below. Which means, not all explanatory variables that significantly influence the crash model on univariate analysis, also have a significant effect on the results of multivariate analysis.

Furthermore, the explanatory variables used in the multivariate analysis results, including: traffic volume, length of roads, number of access points, and the traffic speed. While the explanatory variables which are not eligible in the significance excluded from model building are: variable number of lanes, width of road, road median, number of directions, and the shoulders of the road. So, variables which are simultaneously influential on motorcycle accidents are: traffic volume, length of roads, number of access points, and traffic speed variables.

Table 4. Multivariate Analysis of The Terms

| Parameter | Estimate | Std. Errors | z value | Pr(>|z|) | Significant at 0.05 |
|-----------|----------|-------------|---------|---------|---------------------|
| Constant  | -10.6126 | 1.23897     | -8.265  | <2e-16  | Yes                 |
| FLOW      | 0.44321  | 0.19944     | 2.222   | 0.0263  | Yes                 |
| LR        | 0.75662  | 0.14832     | 5.101   | 3.37e-07| Yes                 |
| AP        | 0.02005  | 0.01010     | 1.985   | 0.0471  | Yes                 |
| SPEED     | 0.06378  | 0.01377     | 4.631   | 3.64e-06| Yes                 |
The estimated value of parameters of all explanatory variables appear in plus mark in Table 4 above. This means that the increase in the rates of traffic volume, length of roads, number of access points, and the speed of traffic is contributing to the rising number of accidents. Furthermore, the final result of the development of prediction models of motorcycle accidents based on the multivariate analysis produced the following formula:

\[ \text{McA} = 0.00002646035 \times \text{Flow}^{0.44321} \times \text{LR}^{0.75662} \times e^{(0.02005 \times \text{AP} + 0.06378 \times \text{Speed})} \]  

(1)

Remark:
- \( \text{McA} \) = the number of motorcycle accidents per year
- \( \text{FLOW} \) = the traffic volume (pcu/hour)
- \( \text{LR} \) = the length of roads (meters)
- \( \text{AP} \) = the number of access points per kilometer
- \( \text{SPEED} \) = the 85 percentile vehicle speed (km/hour)

To examine the final model, comparing the estimated value of the model (fitted value) with the actual data (response) can be done, as shown in Figure 1 below. The results of the model estimation that are close to or in accordance with the actual data of observation results show that the resulting model represents reality occurring on the area.

![Figure 1. The Comparison between Model Estimation with Actual Data](image)

**Interpretation**

To predict the level of motorcycle accident on collector roads in Surabaya, Indonesia, a final model has been developed, as shown in equation 3. It shows that the volume of traffic, length of roads, number of access points, and traffic speeds significantly influence motorcycle accident motors. The result of this study supports the previous studies (Harnen et al. 2006; Xin, 2011; Polus and Cohen 2011; Chengye and Ranjitkar, 2013; Taylor et al. 2002)

The value of the estimated coefficients, both marked with a plus or minus, shows the influence of each explanatory variable to a motorcycle accident (Harnen et al., 2006). The increase in the value of the variables is contributing to the rising number of accidents when the estimated coefficients are marked plus. In contrary, the increase in the value of the variable is contributing to the reduction in the number of accidents when the estimated coefficients are marked minus.

If the traffic volume increases on collector roads, the number of motorcycle accidents will also increase. If the traffic volume doubles, then the model would predict an increase in the number of motorcycle accidents by 26.45%.
The influence of traffic volume in a motorcycle accident is in accordance with the findings reported in previous studies (Xin, 2011; Polus and Cohen 2011; Chengye and Ranjitkar, 2013).

![Figure 2. The Effect of Traffic Flow on Motorcycle Accident](image1)

Each road has a different length. In this model, the difference in the length of roads affects the number of motorcycle accidents. For example, if the length difference between the two streets is twice longer, then the model would predict an increase in the number of motorcycle accidents by 40.8%. The effect of length difference of collector roads on motorcycle accident is in accordance with the findings reported in previous studies (Chengye and Ranjitkar, 2013).

![Figure 3. The Effect of Length of Road on Motorcycle Accident](image2)

The number of access points in each road segment is not the same, there are many roads with many access points but some have few. In this model, the differences in the number of access points per-kilometer among collector roads create the change in the number of motorcycle accidents. The number of motorcycle accidents will increase by 9.54% in every additional 5 access points per-kilometer on collector roads. The influence of the number of access points in a motorcycle accident is in accordance with the findings reported in previous studies (Olesenko, E. and V. Dobromirov, 2013; Tarko et al, 1999; Levinson, 2000; Papayannoulis et al, 2000).

Lastly, if traffic speed increases, the number of motorcycle accidents will also increase. Any increase in the speed of 5 km/h on collector roads, the number of motorcycle accidents will increase by 27.31% (Aaceaah and Salifu, 2011; Taylor et al, 2002).
4. ACKNOWLEDGMENTS

We would like to thank the reviewer for the constructive remarks. We would like to acknowledge the financial support of The Diploma Program of Civil Engineering, Institut Teknologi Sepuluh Nopember, Surabaya, INDONESIA.

5. CONCLUSIONS

Based on the above discussion, it can be concluded, as follows:
1. The final model obtained from the study of the development of prediction models of motorcycle accidents on collector roads is:

\[
\text{McA} = 0.0000246035 \times \text{Flow}^{0.44321} \times \text{LR}^{0.75662} \times e^{(0.02005 \times \text{AP} + 0.06378 \times \text{Speed})}
\]

2. The potential of motorcycle accident on collector roads can be explained in this accident prediction models, that is the accident potential is influenced by the volume of traffic (FLOW), the length of roads (LR), the number of access points (AP), and the traffic speed (SPEED).

3. The increase of motorcycle accidents risk on the collector road is caused by the increase of traffic volume, longer road conditions, the higher number of access points per-mile, and the increase of traffic speed.

This research has produced several findings that provide information to help stakeholders in the field of traffic engineering to solve problems related to accidents on collector roads in the city of Surabaya as well as in other cities. However, the use of this final model may only be applicable in the typical traffic conditions in developing countries such as Indonesia, where the proportion of the total motorcycles is by 60% to 80%.

6. REFERENCES


R-3.0.3 for Windows (32/64 bit), (2014). http://cran.r-project.org/bin/windows/base/old/3.0.3/
PAPER TITLE (90 Characters Max) | The Relationship Between the Use of Traffic Safety Technologies & the Drivers Behavior in Abu Dhabi Highways
---|---

TRACK | |

AUTHOR (Capitalize Family Name) | POSITION | ORGANIZATION | COUNTRY
---|---|---|---
Eng. Musallem Al Junaibi | Researcher, PhD Student at University of Wolverhampton, Traffic Engineering, Traffic Engineering and Roads Safety Department, | Abu Dhabi Police, and University of Wolverhampton | UAE

CO-AUTHOR(S) (Capitalize Family Name) | POSITION | ORGANIZATION | COUNTRY
---|---|---|---
Dr. Panos Georgakis | Lecturer and Researcher, Intelligent Transportation System, Transportation modelling and simulation, Intelligent integrated systems | University of Wolverhampton | UK
Prof. Mushatat, Sabah | Lecturer and Researcher, Professor of Architecture | University of Wolverhampton | UK

E-MAIL (for correspondence) | m80000435@zu.ae

KEYWORDS: Include up to 5 keywords

ABSTRACT:
Various driver behaviors, such as excessive speed, driving under the influence, fatigue and tiredness, use of hand-held mobile devices, and others, have a significant impact on the frequency of occurrence of road crashes. In this respect, road traffic accidents have become a significant public health issue in the Arabian Gulf states. Oil discovered in the region changed the lifestyles in the UAE and resulted in a rise in immigration and population and a subsequent increase in the number of vehicles as well as the expansion of road infrastructure. However, associated behavior was not as quick to change, mostly as behaviors were very much based on the blending of traditional and westernized practices. This caused an increase in the number of serious road traffic accidents as well as an increase in the deaths from these accidents. Previous studies have focused on the issues of aggressive driving behavior as well as driving violations and their correlation with traffic accidents in western countries. However, due to the major differences in driver behavior between UAE and western countries, the adoption of the findings from the aforementioned research studies in UAE is questionable. This paper presents the findings of a study, implemented in Abu Dhabi, which aimed to explore the impact that safety technologies have on drivers' behavior. The study adopted the use of a questionnaire-based survey with drivers using the highways in Abu Dhabi.
The Relationship Between the Use of Traffic Safety Technologies & the Drivers Behavior in Abu Dhabi Highways

Eng. Musallem Al Junaibi, Dr. Panos Georgakis, and Prof. Mushatat, Sabah
University of Wolverhampton, Uk
Email for correspondence: m80000435@zu.ae

1 INTRODUCTION

Road traffic accidents are one of the most common causes of death worldwide. These road accidents have also been known to cause major losses, mostly in terms of property and human lives. The statistics relating to human losses caused by road traffic accidents represent a staggering number. Moreover, injuries caused by these accidents are also significant, ranging from minor injuries to major incapacitation.

The study by Bener and Crundall (2005) was able to indicate that road traffic accident is the second cause of deaths in the UAE. Since then, significant traffic safety improvements have been reached in Abu Dhabi. The number of serious traffic accidents dropped by 33.25% during 2013 compared to 2009, which led to a decline in the number of fatalities by 29.58% and a decline in the serious injuries by 24.02%. This decline was achieved in spite of the increasing number of vehicles (by 45.25%) and licensed drivers (by 41.7%). The statistics of the directorate of Traffic and Patrols at Abu Dhabi Police revealed that the total number of traffic accidents that resulted in injuries and fatalities in 2013 amounted to 3629 incidents compared to 5235 in 2009. In addition, the number of fatalities in 2013 was 288, compared to 409 in 2009.

There are various road user behaviours which often lead to road traffic accidents. Speed is one of the factors which have a significant impact on the occurrence of road crashes. Speed impacts on the risk of crashes and injuries (Simsekoglu, et.al., 2012). The physical parameters of roads including its general environs is said to encourage, or sometimes discourage excessive speed. With higher speeds, crash risk is also higher, especially with lane changes and during overtaking; road users may sometimes inadequately calculate their speed or overestimate their distance to other cars, especially those approaching from the opposite lane (Simsekoglu, et.al., 2012).

The speed of choice is therefore based on various elements, including the driver-related elements like alcohol level, age, and the number of passengers (National Technical University of Athens, 2012). Alcohol impairment can also impact on the severity of the injuries during the crash (Mayrose, 2008). The statistics relating to alcohol and driving related incidents are different for various countries, but all countries agree that alcohol can cause impairment on one’s driving.

Fatigue and sleepiness for drivers has also been seen to impact significantly on the occurrence of road accidents. Elements affecting fatigue and sleepiness may be attributed to long-distance driving, lack of sleep, and disturbed sleeping patterns. Young road users aged 16 to 29 are especially at risk for fatigue and sleepiness leading to road accidents (National Technical University of Athens, 2012).

In recent years, use of hand-held mobile devices significantly affected driver behaviour especially in terms of perception and decision-making (Dragutinovic and Twisk, 2005). Dialing while driving affects the driver’s ability to stay on course. Additionally, based on studies relating to mental loading, driver reaction times increase by 0.5 to 1.5 seconds when the driver is talking to someone on the mobile phone. Studies also indicate that the performance of the driver in terms of securing correct lanes, maintaining distance between vehicles, staying at appropriate speed limits, and judging safe spaces in the traffic are just some of the elements which impact on driver behaviour and which may contribute to the occurrence of road accidents (Dragutinovic and Twisk, 2005). The use of mobile devices while driving, represent a crash risk four times higher as compared to drivers not using mobile devices.

Aside from the failure to use crash helmets, driver and passenger failure to use seatbelts and child restraints have also been considered high risk behaviours. Failure to use these safety devices and restraints have led to fatalities and road crashes with majority of injuries being frontal impacts and often fatal.

In the same aspect road traffic accidents have become a significant public health issue in the Arabian Gulf states. Oil discovered in the region changed the lifestyles in the UAE (Bener and Crundall, 2005). A rise in immigration and population was seen alongside an increase in the number of vehicles as well as the expansion of road constructions. However, associated behaviour was not as quick to change, mostly as behaviours were very much based on the blending of traditional and westernized practices (Bener, 2001). This caused the increase in the number of road traffic accidents as well as an increase in the deaths from these accidents. Previous studies have focused on the issues of aggressive behaviour as well as driving violations and their association with traffic accidents (Mesken, et.al., 2002; Bener, 2001). Most researchers agree that it is not proper to isolate a primary cause for accidents, with early accident studies indicating that 90% of all incidents can be credited to road user behaviour qualities (Kontogiannis, et.al., 2002; Mesken, et.al., 2002). Studies also indicate that casualty rates in the UAE and other Arab countries are higher when compared to developing states due to their higher levels of vehicle ownership (Bener, 2001; Bener, et.al., 2003).
The gravity of the issue therefore implies the importance of studies relating to road accidents, as well as its related risk factors. In general therefore, the activities of road users clearly indicate a significant risk factor. However, factors like age, sex, marital status, training, experience, emotional state, reaction time, vision, and driving speed also have a major role to play in the occurrence of road accidents (Bener and Crundall, 2005). In the UAE, due to the increased pace of its expansion and road developments, road traffic accidents have become a serious public health issue. The impact of the issues can however be significantly decreased where appropriate interventions can be made in order to positively affect road user behaviour.

Based on a 2002 report by Bener and Alwash, a comparison of UAE with other developed nations indicated major differences in terms of driver behaviour. This indicates that cultural and lifestyle qualities are strongly related to a greater risk in the UAE. Assessments of elements of driver behaviour in high income developing states imply that drivers have more dangerous driving habits (Bener, et.al., 2003); moreover, their obedience to traffic rules is poor (Bener and Crundall, 2005). Reports also indicate that for most countries in the Arab region, the seat belt law is greatly ignored (Bener, et.al., 2003; Klenk and Kovacks, 2003). These reports also indicate that social and economic elements may impact on road safety in specific states. The Bener and Crundall (2005) study also suggests that death rates caused by road accidents may reduce significantly the safety conditions in specific countries.

Rowe et al (2010) indicate that a driver improvement plan is focused on developing improvement in driver behaviour. Such plans may include diversion plans for the driver to attend a special course, at the expense of the driver, where the offence is related to dangerous driver behaviour. For example, a course highlighting the dangers of speeding may be warranted for speed related offences where the speed is not grossly excessive but warrants the issue of black points.

The main objective of this paper is to investigate the relationship between the use of traffic safety technologies and the drivers’ behaviour in the light of serious road traffic accidents in Abu Dhabi Highways. Traffic safety technologies refer to systems such as speed cameras, electronic signboards on highways, Low Visibility Motorist Warning System, Intelligent Speed Assistance, GPS, and Adaptive Cruise Control. The study will focus on the effectiveness of the use of enforcement technologies, drivers’ compliance to speed limits, impact of adverse weather conditions, and how technologies can deter traffic offenses.

2 METHODOLOGY

A questionnaire-based survey was used to realize the objective of the study. The survey was implemented using the online survey tool GizmoSurvey, and a number of descriptive statistics were used to analyse the data. The survey aimed to seek Abu Dhabi Emirate highways users' views on the traffic safety technologies. The questions used focused on the investigation of the relationship between the usage of traffic safety technologies and the drivers' behavior. They covered demographic profile for the sample, driver attitudes and behavior, traffic enforcement technologies applications, effectiveness of enforcement technologies, effects of enforcement technologies on specific driver behavior, and deterrence of driver offences. The questionnaire was reviewed and validated by a panel of experts, namely, ten police traffic engineers with more than 5 years experience at traffic and patrols directories in Abu Dhabi. An “intercept survey” was implemented in order to capture the drivers’ responses. This required the interception of drivers, at different locations, while they were carrying out different activities. Such locations included Traffic and Patrols Directories, Traffic Engineering and Road Section in Abu Dhabi, Traffic Engineering Branch in Al Ain, and ADNOC Petrol Stations in Abu Dhabi city and Al Ain. Drivers’ perceptions were analysed based on ranking and likert scales with regard to their opinion on a number of subjects that were presented in the questionnaire.

To estimate the required sample size for the driver surveys for the quantitatively evaluation; the number of driving licenses in Abu Dhabi Emirates was the base for the sample size (1,1151,879 up to end of year 2013; Source: Abu Dhabi Police). The sample size was 665 drivers, which was calculated considering a margin of error of roughly 5% at the 99% confidence level. This means that the 99% confidence interval of a 50/50 split response to a yes/no question is ±5%.

The driver survey was conducted during May and June 2014 at multiple locations. In total, 800 drivers were surveyed. The answers of the respondents to the questionnaire were recorded. After removing the records of respondents who did not complete the questionnaire, the final data set contained 672 surveys, which exceeded the recommended sample size.

3 DATA ANALYSIS AND DISCUSSION

The main dependent variable in this study is the driver behaviour and effectiveness of the traffic enforcement technologies. The independent variables are those related to population characteristics of gender, education level, profession, nationality, age and work status, and weather conditions and types of enforcement technologies.

The following measures are analyzed in the study: Driver knowledge and use of enforcement technologies; Driver behavior and violations; Effects of enforcement technologies on driver behavior; and Effectiveness of various enforcement technologies.
Two types of statistical analysis were employed to analyze and draw conclusions of facts from the collected data. Descriptive statistics were used to derive simple summaries about the sample and the observations that were made. The statistical software, MINITAB, was used to analyze the data.

**Demographic Profile of Drivers:** The demographic information of the respondents, from the total of 672 respondents, is provided in Table 1.

<table>
<thead>
<tr>
<th>Group</th>
<th>Attribute</th>
<th>Number (N)</th>
<th>Percentage (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gender</td>
<td>Male</td>
<td>544</td>
<td>80.95</td>
</tr>
<tr>
<td></td>
<td>Female</td>
<td>128</td>
<td>19.05</td>
</tr>
<tr>
<td>Age</td>
<td>18-21</td>
<td>29</td>
<td>4.32</td>
</tr>
<tr>
<td></td>
<td>22 - 30</td>
<td>216</td>
<td>32.14</td>
</tr>
<tr>
<td></td>
<td>31 - 45</td>
<td>338</td>
<td>50.30</td>
</tr>
<tr>
<td></td>
<td>46-60</td>
<td>83</td>
<td>12.35</td>
</tr>
<tr>
<td></td>
<td>Above 60</td>
<td>6</td>
<td>0.89</td>
</tr>
<tr>
<td>Nationality</td>
<td>UAE</td>
<td>389</td>
<td>57.89</td>
</tr>
<tr>
<td></td>
<td>Arab countries</td>
<td>126</td>
<td>18.75</td>
</tr>
<tr>
<td></td>
<td>Europeans</td>
<td>50</td>
<td>7.44</td>
</tr>
<tr>
<td></td>
<td>Asian countries</td>
<td>94</td>
<td>13.99</td>
</tr>
<tr>
<td></td>
<td>Others</td>
<td>13</td>
<td>1.93</td>
</tr>
<tr>
<td>Education Level</td>
<td>Primary</td>
<td>2</td>
<td>0.30</td>
</tr>
<tr>
<td></td>
<td>Intermediate</td>
<td>16</td>
<td>2.38</td>
</tr>
<tr>
<td></td>
<td>Secondary</td>
<td>153</td>
<td>22.77</td>
</tr>
<tr>
<td></td>
<td>Undergraduate</td>
<td>338</td>
<td>50.30</td>
</tr>
<tr>
<td></td>
<td>Master</td>
<td>140</td>
<td>20.83</td>
</tr>
<tr>
<td></td>
<td>PhD</td>
<td>23</td>
<td>3.42</td>
</tr>
<tr>
<td>Work Status</td>
<td>Student</td>
<td>42</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td>Employed</td>
<td>557</td>
<td>82.89</td>
</tr>
<tr>
<td></td>
<td>Self employed</td>
<td>45</td>
<td>6.70</td>
</tr>
<tr>
<td></td>
<td>Unemployed</td>
<td>23</td>
<td>3.42</td>
</tr>
<tr>
<td></td>
<td>Retired</td>
<td>5</td>
<td>0.74</td>
</tr>
</tbody>
</table>

**Driver Attitudes and Behavior:** A high percentage of respondents (35.7%) use Abu Dhabi Highways on a daily basis followed by those who use the highways 3-4 times a week at 19.2%, while only 4% of the respondents indicated that they don’t use the highways. The frequency of committing traffic violations was investigated and the study found out that the frequency is relatively high amongst the highways commuters with the majority of respondents committing at least a single violation within a year. As found from the study, many drivers (29.5%) commit between 2-5 violations per year, followed by at least 1 violation annually as shown in Figure 1.

![Figure 1. Frequency of violations](attachment:image.png)
The study found out that excessive over speeding is not prevalent amongst drivers, with 83.1% of the respondents not having violations for speeding by more than 60 Km/hr (Lock) in the last three year, and only 5.6% from Abu Dhabi highways drivers have 2 or more violations for the same period. -When questioned on their chances of exceeding the enforcement speed limit of 140km/h, and, only 2.3% did admit that they always exceed, while a considerable number (6.0%) often exceeds the limit. 17.4% and 5.9% do respectively rarely and sometimes exceed the limit, while a significant percentage of respondents (73.0%) never exceeding the enforcement speed limit. In response to low visibility conditions, drivers take varied actions as presented in Figure 2.

![Driver actions under low visibility](image1)

Figure 2. Driver actions under low visibility

A significant number of respondents (40.3%) agreed that the use of speed cameras has the potential of reducing the number of serious accidents with another 25.5% strongly agreeing. While 10.3% were in strong disagreement that speed cameras reduce accidents, a further 8% disagreed. However, 15.8% of the respondents were neutral in opinion. Interestingly, there were fewer number of drivers who were of a strong opinion that speed cameras are only meant to collect fines rather than reducing road accidents with 1.9% and 3.0% strongly agreeing and agreeing respectively. On the contrary, 40.3% and 43.9% of the drivers strongly disagreed and disagreed respectively that the use of speed cameras is to collect fines from drivers, while 10.9% of drivers were neutral.

Most drivers (46.3%) strongly agreed and supported the use of enforcement technologies by the police in managing road accidents with another considerable number (36.5%) agreeing that they are effective in managing road accidents. On the other hand, 2.7% and 1.7% of drivers strongly disagreed and disagreed respectively on the effectiveness of technologies in managing road accidents, while 12.8% remained neutral.

The drivers’ responses on the effectiveness, or relevance of enforcement technologies in terms of accident frequency and severity are shown in Figure 3.

![Effectiveness of speed cameras](image2)

Figure 3. Effectiveness of speed cameras
A significant number of drivers (89.1%) believed that enforcement technologies were sufficient along Abu Dhabi highways with a small percentage of drivers (10.9%) believed that the enforcement technologies are insufficient along Abu Dhabi highways. Figure 4, presents the drivers’ responses on the effect of traffic safety technologies on the drivers’ compliance to traffic calming and changes of speed limit.

![Diagram of drivers' compliance to traffic calming and changing speed limit effects](image)

**Figure 4. Effectiveness of other technologies**

The use of variable message signs (VMS) was found to have a greater influence on drivers’ compliance to maintaining speed limits along the emirate highways with a significant driver population (41.5%) in strong agreement with 44.0% agreeing. Additionally, in raking whether hidden or unhidden cameras are effective over the other in increasing driver’s compliance to speed limits, 16.1% of the drivers were in favour of hidden cameras, 46.3% favored unhidden cameras and 30.7% of the drivers indicated that both have the same potential. The survey also pointed out that reducing the distance between the speed cameras have a higher potential in increasing driver compliance to speed limits along the highways. In that respect, 76.7% of the drivers indicated that it is very likely to completely likely to have positive impacts. As shown in Figure 5, the survey revealed that adverse weather conditions do regularly occur along the Emirate highways. Additionally, the survey found out that the drivers were generally satisfied to strongly satisfied (46.2%) with the current efforts in the use of traffic safety technologies to mitigate the impact of adverse weather conditions on the safety of highways in Abu Dhabi.

![Diagram of drivers' responses on adverse weather effects](image)

**Figure 5. Effects of weather conditions**

The effectiveness of various technologies in mitigating the impacts of adverse weather conditions on road safety performance of Abu Dhabi highways were varied and almost evenly split. In terms of high to very high impacts, additional conventional enforcement led at 56.5% followed by traffic calming enforcement technologies at 51.6%, changing speed limit enforcement technologies (50.9%), more education followed at 50.5%, more variable signs led at 49.8% and sending live information 49.1%. Drivers gave various opinions with regards to the effects on a number of measures on reckless driving. As shown in Figure 6.
The deployment of enforcement cameras on the highways was found to have significant effects on driver behavior. In order of likeliness and with the presence of effective enforcement technologies, Figure 7, shows that it is very, or completely likely that 36.6% of drivers will desist from sudden lane change, 31.3% will desist overtaking in a wrong way, 30% will keep safe distance with leading vehicle, 29.5% will fasten the seat belt, 29.3% will avoid driving dangerously (racing, reckless driving, driving causing danger to others, and sudden swerves), 29.3% will avoid using hand-held mobile phone and 27.7% will drive below the minimum speed limit.

The extent to which traffic safety technologies in Abu Dhabi highways sufficiently deter drivers from committing traffic offences received mixed reactions from drivers. While 62% of drivers believed that they were very to extremely sufficient a small proportion (8.7%) of drivers were of the opinion that they were insufficient to very insufficient.

4 CONCLUSIONS

Serious road traffic accidents impose a burden to society, and therefore it is important to reduce its impact. This paper aimed to investigate the relationship between the use of traffic safety technologies and the drivers behaviour in the light of serious road traffic accidents in Abu Dhabi Highways. This paper examined drivers’ attitudes and behaviour, and found that about 96% of the drivers in Abu Dhabi are using the highways, and 68% from them have one or more violations yearly. Also 5.6% from Abu Dhabi drivers have exceeded the speed limits by more than 60km/h,
while most drivers are using a wrong action while driving under low visibility (Use warning signals). However, 65.8% of respondents strongly agreed, or agreed that the use of speed cameras have a potential of reducing the number of serious accidents. Less than 20% of Abu Dhabi highways drivers believe that the use of enforcement technologies is not relevant to the accident frequency and severity. A small percentage of drivers (10.9%) believed that the enforcement technologies are insufficient to very insufficient along Abu Dhabi highways. Use of variable message signs (VMS) was found to have a greater influence on drivers’ compliance to maintaining speed limits along the emirate highways. Additionally, more drivers were of the opinion that unhidden speed cameras are most effective to increasing driver’s compliance to speed limits. The survey also found out that reducing the distance between the speed cameras has a higher potential in increasing driver compliance to speed limits along the highways. About half of the respondents, support the use of enforcement technologies for mitigating the impacts of adverse weather conditions on Abu Dhabi highways. Furthermore, the survey pointed out that the drivers were generally satisfied to strongly satisfied (46.2%) with the current efforts in the use of traffic safety technologies to mitigate the impact of adverse weather conditions on the safety of highways in Abu Dhabi. The percentage of satisfaction on the current efforts in the use of traffic safety technologies to mitigate the impact of adverse weather conditions on the safety of highways in Abu Dhabi is discouraging. The survey shows that intensifying patrols on the roads, speed cameras and Variable Message Sings, additional drivers training, lowering the speed limit, more CCTV use, and new vehicle safety technologies are effective to address reckless driving behavior in Abu Dhabi highways.

The results of the study revealed the positive effects that traffic safety technologies have on the behavior of the drivers in Abu Dhabi. Therefore, further investment on technological advances in traffic safety (not just speed cameras) can potentially reduce the number of accidents. While automated traffic enforcement technologies are important tools to enhance traffic safety, they should be combined with other speed enforcement methods, education and awareness in order to help reduce the number and severity of collisions on Abu Dhabi’s highways. While cars are increasingly being equipped with new technologies, the use of in-car traffic safety technologies, which aim at increasing or assuring drivers’ compliance with traffic laws, needs to be expanded in Abu Dhabi.

Finally to add more value to this study we can conclude with some recommendations. Immediate sanctions for drivers having exceeded the speed limits by more than 60km/h will be more effective for future compliance. Developing the awareness of the drivers for taking the right actions while driving under low visibility is required. Minimizing the use of the hidden speed cameras is very important to build the feeling that the main aim of using these technologies is to raise the rates of safety in the highways and not to collect fines from the drivers. Expand the use of VMS, CCTV, and new vehicle safety technologies are imperative for improving the traffic safety on the highways.

To add more importance to the relationship between the use of traffic safety technologies and the drivers behavior in Abu Dhabi high ways, future research will include examination of the various factors affecting serious road traffic accidents. Moreover, additional research will study the direct effects of the use of traffic safety technologies on other traffic safety elements.

5 REFERENCES

# The Study of Truck Transport Impacts on Rural Road Network for Future Road Maintenance Improvement Plan

## track

<table>
<thead>
<tr>
<th>AUTHOR (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Koson JANMONTA</td>
<td>Engineer</td>
<td>Department of Rural Roads</td>
<td>Thailand</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CO-AUTHOR(S) (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Koonnamas PUNTHUTAECHA</td>
<td>Engineer</td>
<td>Department of Rural Roads</td>
<td>Thailand</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CO-AUTHOR(S) (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wit RATANACHOT</td>
<td>Engineer</td>
<td>Department of Rural Roads</td>
<td>Thailand</td>
</tr>
</tbody>
</table>

**E-MAIL (for correspondence)**

koson.te@gmail.com

**KEYWORDS:** Rural Road, Maintenance, Truck Route, PMMS

---

**ABSTRACT:**

Truck mode is one of the most important transportation modes in Thailand. With the opening of the ASEAN Economic Community (AEC) in 2015, the transportation of agricultural and industrial products will be increased, especially on the rural roads. This is due to the fact that the rural roads gain direct access to the growing areas. The proper road maintenance plan is therefore needed for the department of rural roads to effectively plan for road maintenance, and fully utilize the limited budget. This study examines the key rural roads in the northeast of Thailand, especially ones that directly join the East-West Economic Corridor. Four maintenance plans are set based on the expected future amount of trucks on the roads. The internal rate of return (IRR) is also performed to rank the worthiness of the road maintenance plan.
ABSTRACT

Truck mode is one of the most important transportation modes in Thailand. With the opening of the ASEAN Economic Community (AEC) in 2015, the transportation of agricultural and industrial products will be increased, especially on the rural roads. This is due to the fact that the rural roads gain direct access to the growing areas. The proper road maintenance plan is therefore needed for the department of rural roads to effectively plan for road maintenance, and fully utilize the limited budget. This study examines the key rural roads in the northeast of Thailand, especially ones that directly join the East-West Economic Corridor. Four maintenance plans are set based on the expected future amount of trucks on the roads. The internal rate of return (IRR) is also performed to rank the worthiness of the road maintenance plan.

1. INTRODUCTION

Road transport under the Department of Rural road’s responsibilities have been increasing steadily due to the ability of accessing to agricultural and industrial areas, including the source and commercial manufacturing facilities such as ports, airports and cargo terminal, etc. Moreover, Thailand and the ASEAN countries are stepping into AEC (ASEAN Economic Community, AEC) in 2015, the transport of goods on the rural roads will be doubled especially the economical route such as the North-South and East-West Economic Corridor routes and Southern Economic Corridor route. This will result dramatically in economic growth in the regions. One of the most important route is the East-West Economic Corridor, which starts from Danang in Vietnam and cut through Laos and Thailand to Mawlamyine in Myanmar. This route is cutting through the provinces in the northeast of Thailand, thus the amount of freights and travels in the future will be expected much higher, especially the increase of tourism and agriculture and industry goods transport. Therefore, it is a must to have a plan to cope with such changes otherwise it could severely damage the roads.

Therefore, the study of truck route network is to develop a model of restructuring and maintenance of rural road, to understand the routes which are used for goods transport and tourism, both domestic and international and to plan and analyse for the development of road network. This will bring up the standards design and maintenance in a proper way to handle the increased of traffic volume in the future.

2. BACKGROUND

Recent research related to the truck route network and road maintenance were revised and shown in Figure 1, which are important factors affecting the development of transport infrastructure and road maintenance. These ideas will be used to prioritise routes that are suitable for developing a truck route network.

Prioritisation is applying Analytic Hierarchy Process (AHP) method by considering the important factors including the number of trucks, connectivity with Asian Highway (AH), and accessing to agricultural and industrial areas. The weighting systems of those various factors are summarized in Figure 2. The prioritised roads will be analysed for the trend of freight transportation in the future to plan for the development of the road network, standard of design and maintenance management.
Figure 1 Recent researches were revised and identified potential routes

- Connectivity with ASEAN Highways
- Traffic Volume
- Industrial and Agricultural Areas
- Tourism
- AEC
- Social and Community

Figure 2 The various factors used in prioritising roads. (Adapted from the Department of Rural Road, 2555)
3. METHODOLOGY

3.1 Prioritising routes and analysing traffic trend in the future

Analytic Hierarchy Process, AHP, is used for prioritising routes which are suitable to be developed for truck route networks. In this case studies, the routes in KhonKhen, Kalasin and Mukdahan which are parts of ASEAN East - West Economic Corridor, were considered and there were 24 selected-routes. There selected routes were examined for the likely traffic volume in the future. The process is shown in Figure 3.

![Diagram showing the process of prioritising routes and analysing traffic trend in the future.]

**Figure 3** Guidelines for the Analysis of trucks route development.
3.2 Decision Making Model for maintenance management.

Traffic volume in the future of the 24 routes will be predicted and used in the decision making model to decide rather it should be maintained or reconstructed. The approach of the decision-making model is shown in Figure 4, this model is more emphasised in how economical of the road is. Also the decision making model for maintenance management used in this study is based on the concept of the Pavement Maintenance Management System (PMMS) which is the system used in the Department of Rural Roads, together with the determination of the level of service of the road. PMMS' concept is shown in Figure 5.

![Diagram](attachment:image.png)

**Figure 4** The decision making model for the maintenance or reconstruction

3.3 Structuring Truck Route Network Development Plan

After considering those models the new structured model was made. The truck route network development plans is created to improve maintenance management system in the Department of Rural Roads, the model can be divided into four categories based on their suitability in economics and engineering.

The process of structuring the truck route network development plan is shown in Figure 6, with the following details.

1. Plan "A" Recovery Maintenance, this plan is for routine and periodic maintenance which is to maintain the road to remain the same condition. This plan is suitable for the road with the truck volume less than 1,500 vehicles / day and traffic volume (AADT) less than 8,000 vehicles / day with the lifetime of 20 years.
2. Plan "B" Recovery Maintenance along with road structure restoration. This plan is initially to follow the maintenance Plan ‘A’, once the traffic volume increased to more than 2,000 vehicles/day then the road structure restoration will be started. This plan is ideal for the road with the number of trucks between 1,500 to 2,000 vehicles/day and the total traffic volume (AADT) less than 8,000 vehicles/day with the lifetime of 20 years.

Figure 5 The decision making for the maintenance or reconstruction in PMMS.

3. Plan “C” Road Structure Restoration and road widening. This plan is to reconstruct the structure of the road and also the road pavement during the truck volume of 2,000 vehicles/day, but when the number of trucks increased to 3,000 vehicles/day, or the total traffic volume (AADT) greater than 8,000 vehicles/day the road widening will be considered. This plan is ideal for the road, with the number of trucks between 2,000 to 3,000 vehicles / day, with traffic volumes less than 8,000 vehicles/day at the beginning of the lifecycle.
4. Plan "D" Upgrade and Road widening (2 to 4 lanes). This plan is suitable for the road with number of trucks more than 3,000 vehicles/day and the traffic volume (AADT) of over 8,000 vehicles/day. This plan will enhance the road capacity to accommodate current and future traffic volume effectively.

Figure 6 Integrated Development Plan of the truck route network
4. RESULTS

After applying the new model of truck route network development to the 24 studied-routes, it was found that there are 14 routes considered in Plan ‘A’. There are 6 routes in Khonkhen Province, 3 routes in Mukdahan Province and 3 routes from Kalasin province.

There is one route is categorised in Plan "B", which is Kalasin-4035, this route is predicted to have an average International Roughness Index (IRI) of 2.77 if it is undergone with the Plan ‘B’.

There are 4 routes are categorised in the Plan "C". Khonkhen-4003 (Intersection with Highway 2109 - Ban Dong Bang) is one of the route and it has a plan to be developed, this will make the IRI at the average of 2.72. Another example Khonkhen-4064 (Intersection with Highway 2109 - Na Fai), this route has identified the development plan to make the IRI at the average of 2.62 which shows in Figure 8. Figure 8 shows the comparison of IRI of the road with and without applying the new model.

![Figure 8 An example of IRI on Khonkhen-4064 under Plan "C"](image)

From the 24 studied-routes, 5 routes are categorise in the Plan "D" including route Khonkhen-1039 (Intersection with Highway 212 – Baan Bang Saiyai), if it is undergone with the plan throughout the scheme, the IRI will be at the average of 2. Also the route Khonkhen-2009 (Intersection with Highway 2 – Baan Samran) has been identified the development plan of the project, this will make the IRI at the average of 2.69 as shown in Figure 9.

Road Structural Design for using in the truck route development plan can be classified into three cases as follows.

1. The road design to renovate the structure is aiming to increase the strength of the structure to accommodate higher traffic volume and reduce maintenance cost. The reconstruction will be done on the original road embankment with 3%-5% additional of crushed stone and cement.

2. The road will be designed to expand the pavement. The purpose is to increase the width of the road from 3m to 3.5m with hard shoulder’s width of 0.5m to 2.5m. Also the new design will increase the strength of the structure to accommodate higher traffic volume.

3. The road design to widen the road, it aims to upgrade the road from two lanes to four lanes to accommodate increased traffic volume. This may require additional land acquisition or expropriation for suitability of 4 lanes road. This road design will need a large budget to progress the project.
Figure 9 An example of IRI on Khonkhen-2009 under Plan "D"

After those 10 routes were classified into Plan "B", "C" and "D" then they were analysed in the economical value of the project. The internal rate of return (IRR) of each route were considered and ranked. After analysing the internal rate of return (IRR) of each selected route then it was compared with the opportunity cost of capital of the World Bank which is 12%, it was found the IRR of those route were more than the World Bank’s opportunity cost except Kalasin-4035 which had a negative IRR.

5. SUMMARY

In summary, the study of the truck route development network for rural road maintenance and restructuring has indicated the routes that are crucial for goods transport and travel both domestically and internationally in the three provinces in the northeast of Thailand. The analysis of the 24 selected routes can be classified into four plans which consists of Plan “A” Recovery Maintenance, Plan ”B” Recovery Maintenance along with road structure restoration, Plan “C” Road Structure Restoration and road widening and Plan "D" Upgrade and Road widening (2 to 4 lanes). Internal rate of return (IRR) was also analysed and found that 9 routes are worth for investment. Department of Rural Road will be applying the study in planning for appropriate maintenance management to handle with increasing traffic volume in the future.

6. REFERENCES

Department of Rural Roads 2012, Feasibility study and design for permanent weighing station on rural roads.

Austroads 1992, Pavement Design: A guide to the structural design of road pavement, AUSTROAD, Sydney

ROADROID CONTINUOUS ROAD CONDITION MONITORING
WITH SMART PHONES

HANS JONES and LARS FORSLOF

Mats Knuts Vag 68, 784 50 Borlange, Sweden
Tel: +46 706997780; Email: hans.jones@roadroid.com
Egnahemsgatan 5, 827 35 Ljusdal, Sweden
Tel: +46 722426620; Email: lars.forslof@roadroid.com

ABSTRACT
Road condition is an important variable to measure in order to decrease road and vehicle operating/maintenance costs, but also to increase ride comfort and traffic safety. By using the built-in vibration sensor in smartphones, it is possible to collect road roughness data which can be an indicator of road condition up to a level of class 2 or 3 [1] in a simple and cost efficient way. Since data collection therefore is possible to be done more frequently one can better monitor roughness changes over time. The continuous data collection can also give early warnings of changes and damage, enable new ways to work in the operational road maintenance management, and can serve as a guide for more accurate surveys for strategic asset management and pavement planning.

Data collection with smartphones will not directly compete with class 1 [1] precision profiles measurements, but instead complement them in a powerful way. As class 1 data is very expensive to collect it cannot be done often, beside this advanced data collection systems also demand complex data analysis and takes long time to deliver the result. With smartphone based data collection it is possible to meet both these challenges. A smartphone based system is also an alternative to class 4 – subjective rating [1], on roads where heavy, complex and expensive equipment is impossible to use, and for bicycle roads. The technology is objective, highly portable, and is simple to use. This gives a powerful support to road inventories, inception reports, tactical planning, program analysis and support maintenance project evaluation.

The Roadroid smartphone solution has two options for roughness data calculation:
1) estimated IRI (eIRI) - based on a Peak and Root Mean Square (RMS) vibration analysis – which is correlated to Swedish laser measurements on paved roads. The setup is fixed but made for three types of cars and is thought to compensate for speed between 20-100 km/h. eIRI is the base for the Roadroid Index (RI) classification of single points and stretches (road links) of the road.
2) calculated IRI (cIRI) - based on the quarter-car simulation (QCS) [1] for sampling during a narrow speed range such as 60-80 km/h. When measuring cIRI, the sensitivity of the device can be calibrated by the operator to a known reference.

Collected data are wirelessly transferred by the operator when needed via a web service to an internet mapping server with spatial filtering functions. The measured data can be aggregated in preferred sections (default 100m), as well as exported to other Geographical Information Systems (GIS) or road management system.

By broadcasting road condition warnings through standards for Intelligent Transportation Systems (ITS) the information could provide new kinds of dynamic and valuable input to automotive navigation systems and digital route guides for special traffic etc.
INTRODUCTION
The International Roughness Index (IRI) [1] is a road roughness index commonly obtained from measured longitudinal road profiles. Since its introduction in 1986, the IRI standard has become commonly used worldwide for evaluating and managing road systems [3]. Measuring road roughness have however been used since early 1900 for expressing road condition and ride quality [2]. Since the end of the 1960s however most road profiling is done with high speed road profiling instruments [3].

The modern traditional techniques for measuring roughness may be categorized as either specially built trucks or wagons with laser scanners, bump-wagons or even manually operated rolling straight edges. Specially built measurement equipment is expensive, due to heavy and complex hardware, low volume of production and the need of sophisticated systems and accessories.

Data gathering and analysis are often time consuming. In the northern hemisphere data collection is typically done during the summer then analyzed and delivered to the maintenance management systems in late autumn. During the winter and spring the road usually faces a continual frost heave/thaw (a very dramatic period in a road’s life with extreme changes in roughness) cycle. The IRI values that were “exact” almost a year ago may now not be the same any longer. Besides, since it’s so expensive to collect and analyze the data, many roads are only covered in one lane direction every 3rd or 4th year.

Smartphone based gathering of roughness data which essentially is a response-type road roughness measuring system (RTRRMS) [1] can be done at a low cost and to monitor changes on a daily basis. For frost and heave issues it can tell when and where it is happening and if the situation is worse than in previous years. It can also be used in the winter to determine the performance of snow-removal and ice-grading. It may advantageously be used in performance based contracts or research on road deterioration, various environmental effects (as heavy rains, flooding etc.) and other adjacent purposes.

It should be mentioned that smartphone based systems like Roadroid might challenge old road knowledge with regard to standards, procedures and existing ways to procure:
- Pavement planners and road engineers knowledge of existing solutions and inputs;
- Research organizations, suppliers and buyers have standards/procedures;
- Organizations have invested time, prestige and huge amounts of money to develop complex data collection and management systems which can achieve exact result.

As described in [1] it is necessary to understand the difference between the four generic classes of road roughness measuring methods in use:
- Class 1 - Precision profiles
- Class 2 - Other profile-metric methods
- Class 3 - IRI estimates from correlation equations
- Class 4 - Subjective ratings and uncalibrated measures

It is natural that skepticism will appear when a class 2/3 smartphone is compared with multi-million dollar class 1 or class 2 instruments as inertial profilers etc. But a smartphone can deliver up-to-date good quality roughness data to a web page within hours, in contrast to an expensive software client with the “exact” class 1 or class 2 data from “last year”.

On the other end of the scale – many road inventories and assessments are made by humans (class 4 subjective ratings) over large areas using only pen and paper.
Smartphone roughness data collection fills a gap between class 1 measurements and class 4 visual inspections. We have lately seen some early adopters of this technology and noticed other steps of development in the market.

THE FIRST PROTOTYPES 2002-2006

The Roadroid team has been working with mobile ITS since the mid-1990s, particularly with mobile data gathering, road weather information and road databases. At TRB in Washington 2001, a Canadian project presented monitoring of speed on timber hauling trucks. It simply assumed that if the speed was low the road quality was poor. Our idea was to add vibration measurements.

Together with the Royal Institute of Technology, a first pilot system was built in 2002-2003. At that time we used a high-resolution accelerometer at the rear axle of a front wheel drive vehicle, connected by cable to a portable PC through a signal conditioner. Two master students built a first prototype using an industrial software system for signal analysis.

![Image](image.png)

Fig. 1. From left 1:st prototype 2002-2003 to right 2:nd prototype, developed 2004-2006.

The initial results were promising and the Swedish National Road Administration (SNRA) financed a R&D project to further develop and validate the prototype with a focus on gravel roads. The system was developed for an embedded Windows car PC with external GPS and GSM capabilities as well as a special A/D board connected to the accelerometer. Also a client based and a web based GIS tool was implemented for viewing the road quality spatially in different colors. A validation between visual inspections and the systems measurements was performed and presented at the Transport Forum in Linköping, 2005. The research was based on 35 segments of 100m which were individually assessed according to 4 road condition classes. A Matlab module analysis was performed on reference samples of specific sections of the 4 condition classes.

A regression analysis was then performed with rules based on:

1) accelerometer amplitude levels
2) RMS (Root Mean Square) algorithms
3) measured vehicle speed and
4) sample data length.

The analysis showed that a single test run with the system properly could classify up to 70% correct compared to an average of subjective visual expert inspections. A single subjective visual expert inspection could however vary much more from the average correct than the systems classification. The method was considered objective with very good repeatability. In 2006 the development stalled. The system was considered relatively cheap and simple to operate at the time (~$7000). In retrospect, it had several limitations; particularly the sensor mounting and cables exposed in the harsh often wet environment under the vehicle chassis. Also the limitations from non-integrated standard components as Windows 98, the car PC, specific cable connections, and to handle the system solution as a whole by the end user.
FURTHER DEVELOPMENT 2010-2011

In 2010, the ideas from 2002-2006 were reviewed. A major technical development was the appearance of smartphones. Literally all peripherals that previously were connected by special cables were now built into a smartphone (accelerometer, GPS, Processor, memory and data communication) and the limitations of certain components were removed by new advances in technology. We knew the answers to some of the questions from 2002-2006, e.g. the basis for signal analysis, the influence of speed and different vehicle characteristics etc. There were however new big questions to solve, such as:

- Was it possible to pick up the “filtered” signals from the vehicle chassis?
- We knew different car models would give different signals, how could we cope?
- Would a lower sampling frequency be enough (100 Hz compared to 512-1024 Hz)?
- Would the smartphone accelerometer sensitivity and range (+/-2 G) be sufficient?
- Would different smartphone models return different measuring values?

We developed an Android application and some test algorithms using the built-in accelerometer signal. The choice of Android rather than iPhone was made considering the open architecture and hardware price/performance relation. We started to sample data on different roads with different types of vehicles, and run over constructed obstacles in 2011.

![Fig. 2 a) left and b) right – Testing the 3:rd prototype in 2011.](image)

We choose the best hardware at the time as reference device (Samsung Galaxy Tab GT P1000). The obstacles were run over by different vehicle types (from small car to large 4WD jeep) a number of times in 6 different speeds: 20, 40, 60, 80, 100 and 120 km/h.

Data were sampled with different devices, both with our algorithms and by collecting the raw accelerometer signal. During the data analysis, we discovered a number of things:

- Differences between car models, especially at low speeds. In the 40-80 km/h range, differences are however limited. The tests gave us a model for how to calculate the speed influence of the signal for three different types or classes of vehicle body.
- Discrepancies between different devices, both for the sampling frequency and the sensitivity of the accelerometer (up to 50%). It is of great importance to know these dynamics to achieve comparable data. A device calibration procedure which can translate the unit characteristics to a known reference device is therefore required.
- It is important to mount the device firmly in a good mounting bracket, preferably in a way that enables the devices camera lens to be directed at the road. Unfortunately few devices have good mounting brackets.

Most importantly: the trials during 2011 showed that usable data could be delivered.
VIEWING OF DATA
Having a device delivering data, we needed a suitable viewer of the information. We created an internet (HTML5) based map tool to present the road condition. The data (which is encrypted) is compressed and sent from the device via a file transfer service to an Amazon web server in the cloud. The uploaded data files from different units are by a hourly routine imported and possibly matched to road links/geometries such as Open Street Map (OSM) [w1] etc. As background map Google Maps with OpenLayers [w2]. The road condition data are divided into 4 different levels for visualization: Green for Good, Yellow for Satisfactory/OK, Red for Unsatisfactory/Not OK and Black for Poor.

![Screenshot of web GIS tool from mid-Sweden.](image)

The mobile app stores a number of data values each second into a CSV file. But to get an overview on a larger scale it is more convenient to use road links with aggregated and averaged data than individual sampled dots. Depending on the spatial road database, there will be many opportunities to refine the data and add attribute information such as road width, traffic volumes, etc. In Sweden we have been using the Swedish national road data base (NVDB) [w3]. Globally we have mostly been using geometries from OSM. The road condition data can be exported in shape format to other systems.

USE OF DATA AND THE ROADROID INDEX
We have been undertaking studies of the International Roughness Index (IRI) and implemented the IRI computations [4] using the quarter-car system (QCS) for our cIRI value. According to [5] there is limited benefit of using the more complex half-car model since the half-car and quarter-car results were nearly identical.

But we also wanted to be able to add data from several measurements over time and compare results over time in a flexible way. We also wanted to automatically generate reports for a specific road and to compare roads with each other and to do comparisons within whole regions.

The solution was to use the percentage of each road class for the individual sampled dots which have been collected within an area. We call this the Roadroid Index (RI). The RI is scalable for a part of a road, a whole road, a city, a region or even the whole world!
Road Condition Change report Q4 - 2012

Gävleborg

Hudiksvall 
Contractor NCC

<table>
<thead>
<tr>
<th>Road no</th>
<th>Traffic Class</th>
<th>Length</th>
<th>Comments</th>
<th>Good</th>
<th>Sat</th>
<th>Usat</th>
<th>Poor</th>
<th>TREND</th>
<th>Good</th>
<th>Sat</th>
<th>Usat</th>
<th>Poor</th>
<th>eIRI avg</th>
</tr>
</thead>
<tbody>
<tr>
<td>E4</td>
<td>14000</td>
<td>1 143</td>
<td></td>
<td>93,9%</td>
<td>4,6%</td>
<td>0,9%</td>
<td>0,5%</td>
<td>3,4%</td>
<td>97,4%</td>
<td>2,0%</td>
<td>0,4%</td>
<td>0,3%</td>
<td>1,8</td>
</tr>
<tr>
<td>83</td>
<td>8300</td>
<td>2 167</td>
<td>Salt road</td>
<td>88,9%</td>
<td>7,4%</td>
<td>2,2%</td>
<td>1,5%</td>
<td>3,3%</td>
<td>85,6%</td>
<td>8,0%</td>
<td>3,2%</td>
<td>3,2%</td>
<td>2,6</td>
</tr>
<tr>
<td>84</td>
<td>7500</td>
<td>2 210</td>
<td>Salt road</td>
<td>90,9%</td>
<td>6,1%</td>
<td>1,7%</td>
<td>1,3%</td>
<td>1,6%</td>
<td>92,5%</td>
<td>4,8%</td>
<td>1,6%</td>
<td>1,1%</td>
<td>2,9</td>
</tr>
<tr>
<td>305</td>
<td>1200</td>
<td>3 105</td>
<td></td>
<td>76,7%</td>
<td>14,4%</td>
<td>5,3%</td>
<td>3,6%</td>
<td>0,6%</td>
<td>77,3%</td>
<td>13,3%</td>
<td>5,2%</td>
<td>4,1%</td>
<td>4,5</td>
</tr>
<tr>
<td>307</td>
<td>900</td>
<td>3 75</td>
<td></td>
<td>93,7%</td>
<td>5,2%</td>
<td>0,7%</td>
<td>0,4%</td>
<td>0,4%</td>
<td>93,3%</td>
<td>5,5%</td>
<td>0,8%</td>
<td>0,4%</td>
<td>3,7</td>
</tr>
<tr>
<td>535</td>
<td>300</td>
<td>3 33</td>
<td>Gravel road</td>
<td>9,1%</td>
<td>23,2%</td>
<td>24,2%</td>
<td>43,4%</td>
<td>7,5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>583</td>
<td>1700</td>
<td>3 89</td>
<td></td>
<td>96,9%</td>
<td>2,6%</td>
<td>0,2%</td>
<td>0,3%</td>
<td>0,0%</td>
<td>96,9%</td>
<td>2,0%</td>
<td>0,6%</td>
<td>0,3%</td>
<td>2,3</td>
</tr>
<tr>
<td>660</td>
<td>1850</td>
<td>3 64</td>
<td></td>
<td>88,6%</td>
<td>8,3%</td>
<td>0,6%</td>
<td>2,5%</td>
<td>0,1%</td>
<td>79,5%</td>
<td>9,7%</td>
<td>4,5%</td>
<td>6,3%</td>
<td>6,7</td>
</tr>
</tbody>
</table>

Fig. 4. Filter, select layers and analyze data in the web tool using the Roadroid Index (RI).

Fig. 5. A Road condition change report using the RI – for performance follow up.

As we want to do continuous monitoring to track development over time we also needed a way to produce reports. Data collection can be made by road guards/officers who are doing visual inspections 1-3 times per week, or by operators such as a newspaper distributor. The RI was very suitable to make reports from the road condition data.

ESTIMATED IRI

We had a promising and scalable index, but knew we needed to correlate this to IRI. To find the correlation we gathered:

1) Data from class 1 (laser beam) IRI measurements, both in 20 meters lengths and the data averaged to road link sections in NVDB [w3].
2) Our average road condition values, both for the matching corresponding 20 meter lengths and the whole road link sections in the NVDB [w3].

By comparing hundreds of road links we established a correlation factor and could estimate an IRI value (eIRI). The initial coefficient of determination (R²) was 0,5 (75%) which was considered satisfactory to proceed. eIRI is used all the way from individual
samples (1 Hz) to an average for a road link. We noticed limitations in speed adaptation, especially on rough pavement surfaces and by mini cars (quite more sensitive than our reference small car).

Research is made by different institutions around the world, as the World Bank, UN, Universities and road engineering companies. Throughout 2014 we have achieved different reports and they mostly show the same result – a correlation of up to 80% reaching IQL3 for eIRI.

- Research by the University of Pretoria 2013 [6] concentrated on finding if Roadroid eIRI was consistent during surveys with varying vehicle conditions as: speed, road path, loads and tyre pressure etc. Conclusion was that Roadroid would be able to produce good results if the mentioned key indicators are standardized.
- Research by the University of Auckland 2013 [7] and [8] was focused on if Roadroid eIRI could represent the roughness felt by motorists to a certain level. Both reports found that the Roadroid application responded to the various road characteristics of the Auckland network in a similar manner to industry accepted systems. To be precise Roadroid had an 81% correlation with laser measurement systems.

CALCULATED IRI
With feedback from organizations and by internal research, calculated IRI (cIRI) was developed. Tests confirms that cIRI - if calibrated correctly can meet the demands for road condition measurements for IQL 2/3. However this needs a stable speed at 60 - 80 km/h. The quarter-car models two swinging weights is simulated with the car body movement as an input. We estimate the car body/frame movement on the accelerometer data and a vehicle calibration variable, which can be adjusted in small steps by the operator.

Validation of sample data and the IRI output has been made with the ProVal (Profile Viewing and AnaLysis) [w4] software. ProVal is the most widely-used, validated and reliable tool for pavement profile analysis according to the developers (The Transtec Group) and their US clients - Federal Highway Administration (FHWA) and the Long Term Pavement Performance Program (LTPP). We notice limited IRI-correlation on roads with rough surfaces (as chipseals/brickroads), but at the same time promising results on gravel roads in Afghanistan and Sweden. From what we have seen, it is a mainly a question of filtering the data correctly to get the correct device motion.
Further work needs to be done, but so far the results points at higher accuracy than the eIRI. As mobile technologies and sensors get more advanced, suitable distance measuring devices between the road and car body chassis might be used in the future.

**DATA AGGREGATOR**

The internet map is a good way to view and quickly analyze the data, but we also needed to get data in sections like 100 meters to produce charts.

So in the web tool, and file details there is a data aggregator that aggregates and averages eIRI and cIRI, speed and altitude (grade) in selectable sections of 25-200m. It also adds date/time, coordinates (latitude, longitude) to each row. The data can be imported to spreadsheets or Road Maintenance Management Systems (RMMS).

**PROFESSIONAL USE 2013-2014**

During 2014 a large scale survey with 12 cars is made by the Swedish automobile association (Motormännén) [w5]. The organization will survey >100.000 kilometers of the Swedish road network to identify and point out road defects. A report will be produced to monitor where the worst roads in Sweden are. The project is partly financed by the Swedish Transport Administration (Trafikverket) [w6].

**USE OF THE BUILT-IN CAMERA**

As most smartphones now have a high quality built-in camera and GPS, Roadroid developed a function to easily take GPS-tagged photos and position them on the map. The images are often of acceptable quality, but are subject to mounting and light conditions.

Fig. 7. A 72 km test run in South Africa – an aggregated plot of eIRI, speed and altitude.
This can be a valuable support for visual inspections, and can also be used to capture dynamic events, such as snow/ice conditions or other maintenance contract issues. We have also tested a high resolution, GPS data action video camera (CONTOUR+2) with good results for more precise and demanding video requirements.

USE ON BICYCLE PATHWAYS
The cities of the world are facing more and more traffic problems. Cycling is one option to commute instead of being stuck in traffic jams. The primary need for cyclists are safe cycle roads but also ride comfort. One important parameter here is the surface roughness. Roadroid has been adapted to a feasible quality monitoring system for bike paths.

Since there is little or none previous work in this field and established road condition standards as IRI are absent, so we use our own road class standard and an 4 level scale index. To get dependable and reliable results we use an approved bicycle trailer, and mount the Roadroid device firmly on the wheels axle. Since many cities not yet have made any inventory of their bike paths the spatial data (lat/long) as input to create the geometries for bicycle pathways.
CONCLUSION

Using smart phones for road surveys is an efficient, scalable, and cost-effective way for road organizations to gather road condition data.

In this paper we have illustrated this with the Roadroid system. The system does not require a phone network connection during the data collection can easily geo-locate data on a global level with sufficient accuracy even down to object levels.

- The estimated IRI reach IQL 3/4 and correlates up to 75-81% compared to IQL1 (laser) devices [7], [8] and [9].
- The calculated IRI can with consistent speeds in 60-80 km/h, and tuned vehicle sensitivity reach higher accuracy (Possibly IQL 2/3)

Although the big benefits comes on a network level, where data needs to be collected for a large road network. Here both the ease to collect lots of data and using the Roadroid index to search and filter by geographic areas comes to its full potential.

Fig 10. Gathering of 1.2 million data point collected by local operators in Myanmar.
REFERENCES

WEB REFERENCES

ACKNOWLEDGEMENTS
The Kartographic Society - Innovation award (Sweden, March 2013)
UN World Summit Award - Global Champion in eGovernance (Abu Dhabi, February 2013)
European Satellite Navigation Competition - Regional winner (Munich, October 2012).
EFFECTIVE AUTOMATED ENFORCEMENT
ESSENTIAL GUIDELINES FOR GOVERNMENT IMPLEMENTATION

Philip J. Wijers
GATSO Traffic Enforcement
Haarlem, The Netherlands
E-mail: p.wijers@gatso.com

ABSTRACT:
An effective road safety strategy needs a balanced approach on engineering, education and enforcement. When motorised mobility starts to increase with economic growth road safety suffers dramatically. Expansion of manual police enforcement is no longer feasible which encourages countries to also incorporate automated traffic enforcement into their road safety strategy. Moreover, the traffic police, as an essential, costly and limited resource should mostly be used for tasks which cannot be automated. This presentation does not present research results and related conclusions. The focus lies on recommendations for policy makers in connection with the concept “enforcement chain”. In particular with automated enforcement (e.g. speed and red light cameras) this chain concept is crucial: if one link is not effective or efficient, enforcement is compromised and road safety is negatively affected. The automated enforcement chain consists of a dozen links from detection and measurement to fine collection and/or court proceedings. It is affected by basic issues such as vehicle registration and readability of license plates. Many counties struggle to get the wide range of multi-disciplinary aspects of the enforcement chain right and are thus not able to optimise their automated enforcement to further improve road safety.

Key bottlenecks in the automated enforcement chain can be found e.g. when
1) the legal and operational framework and its processing capacity are not geared to dealing with automated traffic enforcement and the resulting amounts of traffic violations,
2) the license plate/vehicle owner and/or driving license administration are not in order,
3) inadequate requirements for the legal integrity of evidence and equipment homologation,
4) ticket processing, issuance and fine collection are ineffective and inefficient,
5) authorities argue over ticket revenue and funding of road safety investments e.g. enforcement and processing equipment, software and maintenance,
6) political and administrative support for automated traffic enforcement are wavering,
7) public support suffers because the background, results and road safety benefits of enforcement are not properly communicated through various media.

In case of bottlenecks in the enforcement chain and its legal and operational preconditions the investment in the enforcement infrastructure is, in the best case scenario, not used optimally. In the worst case it could be virtually wasted. Recognising the various actions required to create the proper preconditions and optimise the enforcement chain can prevent this and make automated traffic enforcement what it can be: an effective tool to improve and maintain road safety.

1. INTRODUCTION

When motorised mobility starts to increase with economic growth road safety suffers dramatically. Crash and casualty statistics in many countries illustrate this. The increase in motorisation far outpaces the expansion of infrastructure. Moreover, investment for the lagging infrastructure expansion mostly focuses on extension and widening of the road network, not on incorporating road safety features. Quantity triumphs quality.

A capacity related issue can also be seen in the area of enforcement. Expanding manual traffic enforcement by increasing the number of police officers to cope with the higher traffic volumes and expanded infrastructure is no longer feasible and often not desirable. Especially for speed and red light violations automated enforcement cameras have shown to be very effective from a road safety, operational and cost perspective. This has been confirmed by academic research in various studies. In many developing countries adding automatic speed cameras, to compliment manual enforcement by sometimes subjective local police officers centralises fine revenue collection, reduces ‘fine leakage’ and thus results in a more efficient, fairer and higher fine collection. The traffic police, as an essential, costly and limited resource should be used for tasks which
cannot be automated i.e. checking for alcohol, helmet, seat belt and mobile phone (ab)use. These conditions encourages many countries to incorporate automated traffic enforcement into their road safety strategy. Traffic enforcement with cameras can only be effective if certain preconditions are met and if the automated enforcement chain is effective and efficient. The enforcement chain is a sequence of events and actions from the initial detection of a violation to the final verdict in a court of justice on that violation. In case of an observation by a police officer the enforcement chain is relatively simple. In a court of justice, based on the police oath, the judgement of a police officer on a traffic violation prevails over that of the violator. However, when that judgement is transferred from an officer to a technical device such as a camera various complex issues arise. When not properly resolved these could have a detrimental effect on the enforcement chain and thus on road safety.

This policy paper deals with automated traffic enforcement and the required preconditions and changes to links in the enforcement chain meant to make such enforcement a proper road safety tool. Several countries struggle to make a smooth start with automated enforcement since it involves multi-disciplinary changes which involve various ministries and other (semi-) government organisations, often at different administrative levels. The consequences of not getting the automated enforcement chain and its preconditions right leads, at best, to a sub-optimal investment of public finances and a failure to maximise the road safety benefit of enforcement. At worst, it could lead to a longer term suspension of automated enforcement with a painful legal, administrative or operational repair process, financial losses and a postponed reduction in traffic casualties, injuries and crashes.

This article subsequently deals with the preconditions for automated enforcement, then a step by step description of the enforcement chain links indicating specific bottlenecks. Recommendations are given to resolve these bottlenecks in order to improve the efficiency and efficacy of automated enforcement.

2. PRECONDITIONS

Certain preconditions need to met for automated enforcement to be effective. Some of these conditions e.g. a proper legal framework and an up-to-date vehicle registration, are an absolute must. Others may have a slightly lower priority but nevertheless play a key role in the enforcement chain and thus a successful road safety policy.

2.1 Legal preconditions

When prosecuting offending drivers with automated enforcement equipment governments need to specifically incorporate such enforcement and its operational conditions into its laws and related procedures since automated enforcement is inherently different from manual enforcement by police officers. An important legal starting point for automated enforcement is the choice between owner and driver liability. With owner liability the ultimate responsibility for violations registered with automated enforcement systems lies with the owner of the vehicle. It does not matter if he or she was not driving; the vehicle owner is liable for all the traffic fines. If the owner did not commit the traffic violation it is up to him or her to recover the fine from the offending driver. It also means that an image with license plate on a vehicle together with violation data such as time, location, nature of the violation, etc. is sufficient for prosecution. With driver liability the driver of the violating vehicle is responsible. This means a image or video of a vehicle with a recognisable face of the driver and a license plate is required in order for a violation to be legally valid. Counties with driver liability (e.g. Germany, Switzerland, Denmark, Sweden, Poland and Japan) therefore usually take violation photos from the front of the vehicle. Even with driver liability the license plate is used to identify the owner who is then asked to identify the driver if the owner was not driving at the time of the violation. The choice for owner or driver liability has great implications in various other areas. Prosecuting motorcyclists with a helmet is a problem with driver liability. Managing a demerit point driver license system is more challenging with owner liability.

The European Transport Safety Council (ETSC) states that legal regimes with owner liability contribute to road safety. Processing the huge number of violations resulting from automated enforcement programmes can be done very efficiently and with little or without human intervention by means of ANPR (Automatic Number Plate Recognition) under an owner liability regime. Driver liability is labour intensive and operationally costly due to the matching requirement of the vehicle owner with the actual driver at the time of violation. Moreover, due to e.g. lighting conditions, glare, coatings on car windows or intentional obstruction, recognisability of the driver on an evidence photo may be difficult, which reduces the prosecutability rate of the registered violations. According to data reported from the state Baden Württemberg in driver-liability Germany, two-thirds of the image-based violation cases are stalled (including foreign vehicles and motorcyclists).
Other issues with need attention from a legal perspective include applying administrative law instead of criminal law for relatively common and light traffic offences. There is a considerable risk that the huge number of registered traffic violations (annually around 10 million in the Netherlands) generated by a large automated enforcement camera programme could cripple the court system if they cannot be settled effectively and efficiently by means of administrative fines. With administrative law a violation is legally not pursued further provided a fine or rather administrative sanction is paid. Not paying sanctions in time will not result in a lengthy prosecution through the legal system. However, it leads to rapid increases of the fine based on clearly specified dates. If sanction payment is still not forthcoming it can, for example, be collected by impounding property or high-jacking the violator’s bank account. In the Netherlands a court appeal to an administrative sanction can only be accepted if the sanction amount is paid first. If challenged successfully, it will be refunded. This rule drastically reduced the number of violations processed through the court system. Severe traffic violations (e.g. drunk driving, excessive speeding and violations resulting in crashes) remain subject to criminal law.

Based on the above the recommendation for countries starting with automated enforcement would be to have a legal framework based on owner liability with administrative law to efficiently process light and frequent traffic violations. Create potentially forceful and effective fine collection, which prevents an overloaded judiciary.

2.2 Legal integrity of the evidence

Over the past years legal challenges to the evidence produced caused several camera enforcement programmes in various parts of the world to be terminated due to irregularities with data integrity, the measurement process or its accuracy. Evidence integrity is an essential precondition for the proper functioning of the enforcement chain and refers to two key aspects:

- Accurate, valid and all-inclusive measurements
- Evidence data integrity

Accurate and valid measurements

For the violator at the start of the enforcement chain and eventually also for the judge at the end of the chain there should be absolute certainty that e.g. the BAC (blood alcohol content) or speed of a violation is measured accurately. When violations are challenged judges and public prosecutors need to be able to refer to independent objective references which confirm the accuracy of measurements. Such reference is provided by an official certificate after a type approval procedure, issued by a government or authorised institute involved in testing equipment for measurement accuracy (e.g. PTB (Germany), LNE (France), NMi Certin (Netherlands) or METAS (Switzerland)). Regular (often annual) calibration is also legally required to confirm that measurements remain accurate and valid. This calibration is especially important for speed enforcement and BAC testing equipment. The use of enforcement equipment without certification and regular calibration is often not allowed and such measurements are not considered legally valid. For the judiciary but also for the violator these procedures guarantee that measurements are impartial, reliable and accurate and prove that an undisputable violation occurred. A low success rate for legal challenges is important since it discourages future disputes and eases the workload of the judiciary.

Besides accuracy, measurement institutes dealing with type approval increasingly require e.g. a 97% vehicle detection rate and that 95% of passing vehicles are actually speed measured. Type approval tests take place in laboratories, ‘in the field’ and after actual installation. Test data are collected under night/day, various seasonal and weather conditions in practical tests spread over several days.

Besides measurement accuracy and detection rate, the measurement validity needs to be tested. Invalid readings (e.g. a car overtaken by a speeding vehicle is registered as violator) should be eliminated by either not registering such evidence or marking the registered evidence as invalid. Inaccurate and invalid measurements may cause unjust violation notices, driving license suspensions and revocations (especially damaging for professional drivers), unwarranted fines and undermine the credibility of enforcement. This in turn hampers road safety.

Evidence data integrity

The evidence of a violation should be registered permanently and securely. In the past, with analogue wet film as evidence this was not an issue since there was only one original hardcopy negative which included all violation data. Tampering with the photo negative clearly leaves marks. However, this changed with the introduction of digital enforcement cameras and the registration of digital image, video and violation data. Thorough tests are required to confirm that the digital evidence is secured with data encryption, authentication and integrity standards during data registration, transfer, storage and retrieval. Once registered there needs to
be absolute certainty that violation data are confidential, secure, permanent, read-only and rendered invalid when changed. Type approval institutes check for various aspects of data integrity as a key condition for equipment certification.

For governments considering or already using automated enforcement equipment this paragraph confirms the legal importance of proper type approval and calibration procedures.

2.3. Operational preconditions

Since automated enforcement is based on recognising vehicles by means of license plates, an accurate up-to-date vehicle and owner registration database is a prerequisite for such enforcement. This also applies to driver liability countries, since the initial contact point is always the vehicle owner. Another requirement is a driving license database to verify the validity of driving authorisations and, when applicable, the related demerit points of the driver in question. This is particularly relevant in driver liability countries. Demerit points are subtracted in connection with the severity of a violation and may result in driving license suspensions.

Arranging proper access authorisations to such databases by authorities involved in the enforcement chain is crucial.

A complicating issue, also in light legal equality, is how to deal with violations involving foreign vehicles. It is considered unjust if vehicles with foreign license plates can speed and escape fines because the vehicle owner cannot be traced. In France an average of 25% of the speeding violations is committed by foreign vehicles. This figure goes up to 40-50% during holidays and other peak travel periods. This is issue was resolved in November 2013 with the EU Cross Border Enforcement (CBE) Directive. This allows access to the various national vehicle registration databases between most EU countries. Under the CBE violations for eight types of road safety related traffic violations can be sent to the owner in the language of the country of the violating vehicle. For many countries this is not a viable option. In such cases the license plate of the violating vehicle will have to be checked against a database with open unpaid citations at border crossings. Fine payment has to take place on the spot as a condition to enter or leave the country.

In many countries license plate databases are much more than a link between the license plate, owner and address. They also include other data on vehicle type, length, width, weight, axles, engine size and type, exhaust systems and emissions, etc. This gives authorities traffic management opportunities. The City of Amsterdam effectively keeps older trucks and buses with polluting diesel engines out of the city centre with an environmental zone system based on engine and particle filter information stored in the national vehicle registration database. Violating vehicles are fined 230 euro’s. Dedicated bus lanes, stations squares, central city areas and shopping streets can also be kept car or truck free in a similar way.

2.4. Political support and Funding

Economic growth often follows a pattern whereby initially the development priority is on expansion of infrastructure. Political attention to road safety often comes later when the increased motorised mobility further facilitated by the focus on infrastructure leads to socially unacceptably high casualty and injury figures. The Commission for Global Road Safety headed by Lord Robinson cites a low political priority of road safety in low and middle income countries. It seems many politicians and bureaucrats see extending and widening a road network as an investment benefitting economic development, whereas improving the safety of it is seen as cost. The World Bank also stresses infrastructure improvement, capacity building to address systemic weaknesses and attention to vulnerable road users as key policies to improve road safety in low and middle income countries. Low cost measures which are known to be very effective include speed humps, zebra crossings, helmets and effectively separating fast and slow traffic. Road safety in many emerging countries could benefit by massively implementing such value-for-money measures.

The huge human and material losses should justify politicians’ attention at an earlier stage, but they also make economic sense. Road safety losses are estimated at up to 5% of GDP in some low income countries. Still, road safety is not an election winner and it is next to impossible to score electorally by vowing to improve road safety by increasing automated enforcement. Some politicians find that backing higher speed limits (for some reason equated with mobility) and reducing traffic jams, makes electorally more sense than backing road safety and effective enforcement. However, there are objective reasons for politicians to advocate and promote more effective enforcement. The connection between automated enforcement and the benefits to road safety is tested and confirmed in many academic publications. The Economist stated: ‘The cost of the cameras was repaid fivefold within a year in accident reduction and savings in medical treatment’. Moreover, lower speeds reduce pollution, noise levels, fuel consumption and thus add to the quality of life.

Automated enforcement requires funding i.e. considerable investment in human resources, equipment, systems and publicity, scarce items in many low and medium income countries. It also needs amendments to
several laws, involving complex negotiations between several ministries and other government bodies. Since political attention for road safety and automated enforcement does not come naturally it needs constant publicity and advocacy to create awareness and strong political and public support. In turn this should motivate voters and thus politicians to further improve road safety, which often happens in high income countries especially at a community level e.g. due to child safety and noise and pollution concerns. France remains one of the best recent examples of the positive effects that strong political support can have on road safety. Remarkable progress has been made since 2002, a landmark year in French road safety policy. In the wake of a personal statement by President Jacques Chirac, the Interdepartmental Road Safety Committee (CISR) adopted a comprehensive plan in December 2002 to install a nationwide network of automatic speed enforcement systems and to fully automate the enforcement chain for driving offences, in particular for speeding offences. Effective curtailment of speeding by means of increased mobile and in-vehicle police enforcement, the nationwide enforcement camera project (roughly 4,150 fixed and mobile speed cameras in 2013), the efficient processing of the violations notices and the effective fine collection process is believed to be the principal reason for the considerable drop in the number of road deaths and injuries which was the French government’s aim. Other road safety risks such as drunk driving were also successfully addressed. By also adding unmarked police vehicles equipped with speed cameras which automatically register both receding and approaching speed violators, the French government has successfully maximised the subjective chance of being apprehended as experienced by the French driving public. This new enforcement regime drastically changed the driving mentality of French. The 2013 IRTAD report stated that between 2002 and 2010 the average speed in France decreased by 10% and the speed violation rate decreased from 60% in 2002 to 33% in 2010. It is estimated that this contributed to saving 11 000 lives between 2003 and 2010. The comprehensive speed enforcement strategy in France is extensively documented by Mr. Laurent Carnis of IFSTTAR.

2.5. Publicity and advocacy to promote awareness, public and political support
In many countries strict enforcement, often with lacking publicity, may create a serious backlash among the public, thus the electorate and in various media. Examples of this are well-known in e.g. Germany, the UK and the Netherlands. The UK therefore uses detailed documentation on criteria for the location, operation, visibility and signage announcing speed cameras. Automated enforcement as a standalone activity runs a serious risk of being seen as a tax or a limit on freedom. This is actively encouraged by various media e.g. by reports on huge amount of collected fines. The fact that such revenues often benefit the ministry of finance or the general municipal account does not help. Showing a financial relationship with road safety or infrastructure improvements would.

Few people object to enforcement in their own neighbourhood and in front of the school of their children, the YIMBY (Yes In My Back Yard) effect. However, further away from home, e.g. while trying to be in time for a meeting, the mood changes and a negative perception towards enforcement often takes over.
Continuous communication on enforcement benefits by means of publicity, education and advocacy to create better road safety awareness and more public and political support for enforcement is essential for its long term success. Regular publicity campaigns about the reasons for and results of enforcement programmes are a crucial part of a balanced road safety strategy. Governments implementing automatic enforcement need to take notice of the concept ‘no enforcement without publicity’ since a negative public perception against enforcement is hard to change.

3. THE ENFORCEMENT CHAIN

Enforcement works best when the violator is notified quickly of a violation and if the punishment, mostly the payment of a fine, is perceived as high enough to act as a deterrent. Administratively fine collection can be a difficult and laborious process if the violator is not willing to pay. Therefore, it has to be absolutely clear to the violator that a true violation took place, there is no escape from such fine and that the will continue to increase with payment delay and during the appeal process. Eventually, the violator will be ordered to appear before a judge at the end of the enforcement chain and that the escalated and rightfully issued fine will be collected by whatever legal means possible.

News of an ineffective enforcement chain travels fast. If violators can afford to skip punishment, the number of violations will increase rapidly, enforcement losses effect and road safety suffers. Apart from this issue, ineffectiveness and inefficiencies can appear in several other links of the enforcement chain mostly with negative effects on road safety. It is therefore important to analyse each link of this chain for potential bottlenecks. Governments starting with automated traffic enforcement have the benefit to learn from the experiences of other countries.
The automated enforcement chain consists of the following links which will be dealt with in the paragraphs below.

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Detect</td>
<td>7. Process evidence, issue and send ticket</td>
</tr>
<tr>
<td>2. Measure</td>
<td>8. Receipt of ticket</td>
</tr>
<tr>
<td>3. Decide on violation</td>
<td>9. Provide evidence upon violator's request</td>
</tr>
<tr>
<td>4. Register</td>
<td>10. Collect fine</td>
</tr>
<tr>
<td>5. Transfer</td>
<td>11. Remind violating party</td>
</tr>
<tr>
<td>6. Store evidence</td>
<td>12. Court</td>
</tr>
</tbody>
</table>

3.1 Detect
A high detection rate is a key item at the start of the enforcement chain. Identifying all passing vehicles is important since only then all potentially speeding vehicles can be caught. If only a low percentage of violators are detected and others escape, feelings of injustice arise which could affect the public support for enforcement. This requires high detection rates.

All detection methods have their advantages and disadvantages. High detection rates are possible with reliable and durable detection methods such as inductive loops or radar. If installed properly (loops) or mounted sufficiently high (radar) the overall disadvantages of these detection methods (e.g. weather conditions, wear and tear, dirt from passing vehicles or occlusion (blockage of a violating vehicle by another vehicle)) are quite limited. Although accurate, other methods such as video and piezo, laser and low mounted radar all suffer disadvantages with light conditions, weather, occlusion by other vehicles, soiling, durability or sensitivity to damage, which reduce detection rates and thus legal equality.

3.2 Measure
After detection of a vehicle its speed needs to be measured. Obviously, this needs to be done correctly with a legally pre-defined accuracy (ref. §2.2 – type approval and calibration). Is should be noted that the increasingly popular average speed enforcement systems (section control) calculate the speed based on accurate distance and time measurement and not by measuring spot speeds. Measurement is not relevant for red light enforcement. Detecting the passage of a vehicle during a red light phase is crucial here and can be observed visually since both the vehicle and traffic light can be seen in images and/or video.

3.3 Decide on violation
If a person or vehicle is measured to exceed a legally set limit in traffic this should result in a violation. In case of speeding, deciding on a violation is based on the comparison of two speed values by the enforcement camera i.e. the speed limit and the registered speed. Also here all violating vehicles should be treated in the same way. When enforcement equipment misses and fails to decide on speeding vehicles this will result in legal inequality. On the other hand, the same applies to vehicles which were registered as speeding and violating but were not. Proper homologation and calibration procedures will prevent such issues.

With new radar technology and the higher data processing capacity of the latest enforcement cameras concurrent violations can also be handled. This means that accurate decisions accompanied by solid evidence can be taken of e.g. three vehicles speeding or going through a red light at the same trigger point at an intersection.

3.4. Register
After a positive decision on a violation the image and data for that violation need to be registered real-time in a correct, secure and unchangeable way. When properly registering evidence data the use of cryptography is vital. Cryptography includes encryption, authentication and integrity. This allows for respectively 1) multiple reads only by authorised personnel (encryption), 2) data originating from the right source (authentication) and 3) no tampering with data by anyone (integrity). Records of non-violating vehicles need to be discarded, but depending on the country may also be kept longer e.g. for security or criminal investigations. Here a related issue comes up. How long can data of non-violating vehicles be kept by the violation camera? In light of privacy and security, which are often conflicting in terms of storage demands, such issues need to be legally defined.

In Germany a privacy issue prevents the implementation of average speed enforcement technology. This is based on the argument that data of innocent drivers cannot be registered, which means data registered at the first camera of a section control system would violate the strict German privacy regulations since it is not know if a driver passing the first camera is going to violate the speed, which can only be decided by a second camera.
Often the license plate data from the registered image file are already processed and read in the enforcement camera by ANPR software and are digitally included in the evidence file. The result of the reading of licence plates from the image is accompanied by a figure indicating confidence level which specifies the certainty of the registered reading. Confidence levels below a predetermined threshold will need additional processing, and may require manual processing which is costly and reduces operational efficiency. In France and the Netherlands with fully automated processing from violation registration to the issuance of a citation an high ‘autoratio’ requirement is crucial. An autoratio of e.g. 90%, means that more than 90% of licence plates of the violating vehicles need to be read. Within this 90% less than 1 licence plate of 10,000 vehicles can be read erroneously. Such requirements put high demands on the readability of license plates, image quality of the enforcement camera and the ANPR software and processing. Governments aiming for high processing efficiency should require high detection and read ratios and low error ratios.

In owner liability countries license plate quality and readability affect the efficiency of automated violation processing. The clarity, uniformity and readability of a license plate affect the quality of the registered evidence. For instance large black letters on reflective white or yellow plates with a predictable uniform syntax and font get the highest ANPR rates. In owner liability countries this will cut processing costs significantly. Some countries (e.g. South-Korea and the Netherlands) have changed the license plate format, material, font and/or syntax to accommodate for this. The AAMVA (Association of American Motor Vehicle Administrators) issued a report with detailed recommendations on various aspects of license plate design to benefit readability for more effective enforcement and homeland security.

3.5. Transfer
After registration the evidence data need to be transferred from the enforcement camera to a central server which, depending on the organisation, can be located with the police, municipality, central or regional government organisation or even with a private company. Transfer of these data files should be subject to the above mentioned cryptography conditions. Such data can be transferred by means of a fixed data line, WiFi or 3G or 4G broadband connection. Depending on the authorised procedures, some countries opt for manually exchanging data storage devices (e.g. hard disks, memory cards or USB sticks) similar to exchanging wet film cartridges with analogue enforcement cameras. This obviously adds to processing costs. Moreover, lose storage devices may get damaged or lost.

3.6. Storage
Violation evidence data are transferred to a secure data storage location where they need to be kept for longer term storage (also ref. 3.4.) to issue citations but also for future reference by either the violator, police, public prosecutor or judge. Such data need to be properly backed up or mirrored at other locations to be secure in case of calamities.

3.7. Process evidence, issue and send ticket
The evidence, which consists of image(s) (increasingly also video) and violation data, requires further processing in the back office to prepare tickets for issuing. There are considerable variations in the operations and functionalities of back offices depending on the legal framework and process architecture. License plates in violation images may need to be read if they have not been processed in the violation camera. Depending on the process license plate data may need to be keyed in or confirmed visually by an authorised person, even though the error rate of this manual procedure is considerably higher than automated processing with ANPR. Most driver liability countries need to identify the driver in the image and have to process all tickets manually. Lighting conditions of the license plates and drivers faces are quite different and need to be adjusted to produce credible evidence. Due to these requirements prosecutability rates for driver liability countries are considerably lower than for owner liability countries. The opposite applies to processing costs per ticket. However, new technology may allow for more automated processing of front photography images in driver liability jurisdictions.

Matching license plates with the owner’s addresses by accessing the vehicle registration database and matching the violation with a fine or penalty are key processes taking place in the back office. Several countries struggle to build and maintain an accessible, correct and up to date registration database. If a vehicle is sold and gets a new owner at say 15:46 on a given day, it needs to be registered as such. Inaccuracies in this database will result in fines for the wrong owner or driver which in turn affects public support for enforcement. When the owner’s address is found and the violation data matched with the legally set fine for the violation in question the citation can be issued and sent.

As mentioned above a combination of owner liability, administrative law, good license plates, high quality cameras and images and effective ANPR software allow countries like France and the Netherlands to annually
process millions of violations from automated enforcement cameras with limited or no human intervention from one national citation processing and fine collection centre. However, when not managed properly these huge volumes of incoming evidence files from cameras can also create major operational bottlenecks which can affect or even shut down enforcement programmes thus hampering road safety. Various media in Belgium reported recently that their average speed control systems had to be turned off at certain times since more violations were registered than could be handled operationally. This resulted in a backlog of roughly 23,000 unprocessed violations. Backlogs which continue to grow tend to run the risk of being dismissed, which sets a serious precedent, creates expectations among drivers for more dismissals and affects the credibility of road safety enforcement.

3.8. Receipt of ticket
After sending out a citation by means of a letter it needs to be received by the owner of the violating vehicle. This all sounds quite logical but governments need to arrange that ‘not receiving’ such notice or letter will not be an acceptable reason for ignoring the violation notice and thus for crippling this link of the enforcement chain. Depending on the quality of the postal delivery, registered mail may be considered but in most countries regular mail is used. However, this means a legal and procedural arrangement needs to be made which implies that sending a violation notice implies receipt of the same. This again stresses the necessity of an accurate vehicle owner registration database obviously including the current owner’s addresses as an essential requirement for effective enforcement.

Research has shown that the earlier the driver is confronted with a violation the better the effect of enforcement is for correcting driving behaviour. This means governments need to organise efficient processing facilities. In owner liability countries technology allows for an almost immediate notice of a violation by means of and e-mail or SMS text message if the owner opts in to have an e-mail address or mobile phone number registered in the vehicle owner registration database.

3.9. Provide evidence upon violator’s request
Many vehicle owners who receive a citation may want to obtain more information, i.e. see the evidence photo with the violation data such as speed, location, time, etc. Such information can be sent by mail or offered by means of internet access. In countries with driver liability and front photography the passenger side is blocked out for privacy reasons.

When providing violation evidence over the internet, other relevant information which further backs this evidence can also be offered. This could include data on the homologation and calibration of the relevant enforcement equipment or anonymous credentials and training data of the police officer in question. After reviewing the various pieces of solid evidence over the internet the violator can be given the option to move to a fine payment function or objection module.

A key aspect which should be presented to a violator is targeted publicity on why enforcement takes place, and more specifically, why on the violation location. For example: “The E37 road stretch where speeding with your vehicle was registered is speed enforced since 3 casualties, 4 injuries and 7 crashes occurred here in the past 5 years”.

3.10. Collect fine
Similar to sending the citation, fine collection should be effectuated as early as possible to maximise the effect of enforcement as a road safety instrument. To encourage timely payment the citation should clearly specify the fine escalation scheme and collection process should the violator not settle the fine within the set due date.

Some countries (e.g. Germany) work with an initial warning or a transaction proposal with a relatively low fine for a minor violation which has to be paid quickly. Late payment initiates a higher fee and a more formal legal process.

Of the roughly 10.3 million fines issued in 2013 the Dutch Central Fine Collection Agency (CJIB) collected 97,5% of the fines within one year. In France the Centre National de Traitement (CNT), a similar centre in Rennes, collected 80% of the 4,5 million issued fines within 45 days in 2010. Most of the violators in France are paid over the internet.

3.11. Remind violating party
As part of an efficient fine collection process and to maintain the efficacy of the enforcement chain payment reminders need to be sent automatically to overdue violators. In the Netherlands the traffic fine is increased by 50% with a first payment reminder if the initial due date of eight weeks is missed. A second reminder doubles the initial fine if the first reminder did not result in payment. If a second reminder is not effective coercion methods will be implemented. This out of court coercion process should be well-defined legally and
procedurally. Coercion methods in EU countries include: using a bailiff, withdrawing fines directly from bank accounts or employer (Attachment of Earnings Order), seizing of tax credits, impounding vehicles, invalidating driving licenses, etc. In several countries vehicles owned by drivers with overdue fines can only pass their periodic motor vehicle inspection or drivers license renewal if all traffic fines are paid. Forced fine payment can also occur during border or airport identification checks when going or returning from a trip abroad.

3.12. Court
To conclude the automated enforcement chain as the final link and if all the above fails, the violator should know that a judge will ultimately cast a verdict at the end of the chain. The public prosecutor and judge need to be supported in their judgement by type approval and calibration certificates of enforcement equipment issued by independent, often government related organisations as referred to earlier. In order to prevent backlogs and subsequent acquittals due to court overload, the preceding links of the automated enforcement chain should be organised and managed in such a way that the number of violation cases ending up in court should be kept to the absolute minimum.

4. CONCLUSION
Governments starting with or intensifying automatic enforcement should carefully consider the full scope and implications of this project, which is easily underestimated. Successfully implementing automated enforcement needs efficiency and efficacy of the entire enforcement chain and compliance with essential legal and operational preconditions. However, enforcement can never work effectively as a stand-alone activity. Only in concert with continuous communication, education, publicity and advocacy to promote public understanding and to maintain political support can automated enforcement contribute to its ultimate purpose: improving road safety. It should also be noted that besides enforcement there are several other ways to effectively manage speed. Various infrastructure but also education and in-vehicle technology related measures have shown to be quite effective.

REFERENCES


ABSTRACT

Misled the driver in one vehicle road causing buildup in one segment, while other parts are still empty. This occurs because the communication between the driver with the road is still lacking. Static signs as a means of communication with the driver is still not enough to communicate with the driver. IRE have developed systems to date information to road users is through electronic signs and this system has been tested in the city Bandung. The goals of this research is to determine how much influence traffic condition information through electronic signs to the buildup of current in one segment. The technique used in the analysis is to calculate the change in current in one segment of the road compared to the questionnaire to the user path. From the results of the study proved that there is a significant effect that a buildup of vehicles on roads that were reviewed diminished after the information about traffic conditions.
VARIABLE MESSAGE SIGNS AS A SOLUTION TO OVERCOME CONGESTION IN URBAN ROAD
Disi Mochamad Hanafiah¹, Sri Bintang Pamungkas²
Pusat Penelitian dan Pengembangan Jalan dan Jembatan
Balai Lalulintas dan Lingkungan Jalan
Kementerian Pekerjaan Umum
Jl. A.H Nasution no.264 Bandung
Pos-el: disi.hanafiah@pusjatan.pu.go.id
bintang.pamungkas@pusjatan.pu.go.id

ABSTRAK
Misled the driver in one vehicle road causing buildup in one segment, while other parts are still empty. This occurs because the communication between the driver with the road is still lacking. Static signs as a means of communication with the driver is still not enough to communicate with the driver. IRE have developed systems to date information to road users is through electronic signs and this system has been tested in the city Bandung. The goals of this research is to determine how much influence traffic condition information through electronic signs to the buildup of current in one segment. The technique used in the analysis is to calculate the change in current in one segment of the road compared to the questionnaire to the user path. From the results of the study proved that there is a significant effect that a buildup of vehicles on roads that were reviewed diminished after the information about traffic conditions.

Keywords: Congestion, electronic signs, reduction of congestion

INTRODUCTION
The transportation problems such as an accident and the traffic jam are increase as a result of the fast growth of a motor vehicle that is not supported by adequate road infrastructure and traffic management. This is proven by the development of a vehicle in Indonesia based on data from the central bureau of statistics from year 1987-2012 that the development of a vehicle in Indonesia develops very rapidly where tax increase reached 1082,4 % that is from 7.981.480 vehicles in 1987 being 94.373.324 vehicle on year 2012. While every year national road we have increased by only 7 percent (%), whereas the increasing of another country is 17-18 percent. Bandung is one of the examples that represent the traffic problems that happening in many big cities in Indonesia. One of the problems from the limited road network development of big cities in Indonesia is that the system of road network in Indonesia has yet to be utilized optimally, it is aggravating the traffic jam that occurred in the major cities in Indonesia. One example of the not optimized system of the road network in Indonesia caused by the information about the condition of the road network in the vicinity could not be accessed by the users of the road.
One of the solutions to reduce traffic jam is by applying a smart transportation system or Intelligent Transport Systems (ITS). One of the application of ITS is variable message sign (VMS). VMS used to give various information to the users of a highway such as information about the average speed of the flow of traffic on a given road, route information and other traffic information traffic. Pusjatan has made ITS technology which the physical form is VMS has been implemented in Jl. Dr. Junjunan in Bandung. Traffic info that conveyed through VMS is about the current condition, because the censorship and processors technology that used is capable to conveying recent traffic condition information. To find out the benefits or evaluation of these VMS, Pusjatan Team do a survey of the distribution of quesioneer by following the methods of the Wisconsin Department of Trasportation Survey regarding the drivers survey. This paper aims to find out how big the influence of traffic signs information condition through electronic or vms to the current traffic flow accumulation in one of the segments reviewed.

METHOD OF STUDY

Location

The study location for this activity is the VMS physical model that attached at Dr. Junjunan Street of Bandung. The site was chosen for the deployment of VMS that provide traffic information conditions from toll road majority users toward Gasibu and useful for drivers to choose the route of their trip whether to choose the lane roads Gasibu or other alternative. In more obvious form of physical model of VMS which has installed at Jl. Dr. Junjunan is as seen below.
Data Collection Techniques

Data technique used to analyze this paper use two methods. First methods used is with spreading a questionnaire to the road users who passes this road every day reviewed with the purpose to know a response from road users against installed VMS in terms of installing, location, font type and information delivery. But that will be shown in this paper is the influence of VMS to users of the road, so just a few questions that will be linked in this paper. The second method is to do a survey counting traffic before VMS functioned and after VMS functioned for at least 1 month.

Data analysis

At this stage of the analysis, the subject of the analysis is the result of counting traffic before and after the VMS are enabled. Two from that data is made a graph and viewed the rate of change that occurs from the amount of traffic flow that passes through the roads Pasopati direction to Gasibu at the morning rush hours and afternoon busy hours. Results from these comparisons are later linked with the results of a questionnaire whether there is a positive influence or no influence at all for the traffic flow.

RESULT AND DISCUSSION

Questionnaire Result

Distribution of the questionnaire done along roads that are reviewed or along the road of Dr. Junjunan. The Target respondents were receiving a questionnaire form was the daily road users that always use the roads for daily activities. Target respondents that we get are, the taxi drivers, university students, employees and students who each day using the roads for their daily activities. While the spread of the questionnaire refer to test the adequacy of statistical data based on the amount of daily traffic average on the road, so the number of questionnaires distributed is sufficient meet the expected targets. From the results of the questionnaires distributed, some of the questions posed to be able to know the influence of the VMS to the road users, especially the driver. From the results of this questionnaire obtained some results as seen in picture 2 below.
From the results of this questionnaire respondents most answered questions with the answers above are seen reaching 58 people, followed by enough visible 45 people and highly visible only 23 people. From the respondents answer, the answers is quite varied with the highest percentage of answer looks with the percentage reached 45%.

The next question is about clarity of the information conveyed through VMS for road users can be seen from figure 3 below.

Most respondents chose to enquiry above is quite readable reached 47 people respondents or approximately 36% of the total respondents while choosing answers read reached 24%, and very
readable reached 32%.

For further questions regarding the perception of the information conveyed through the VMS for the users of the road can be seen in Picture 4 below.

**Figure 4.** Perception of the information conveyed through the VMS for road users

Most respondents chose to answer the question above is understood by the number of respondents who choose to reach 109 people or approximately 84% of the respondents answered understand the information presented by VMS.

For further questions regarding the influence of VMS information against the selection of alternative lanes for the users of the road can be seen in Picture 5 below.

**Figure 5.** The influence of VMS information for the selection of alternative lane for road users
For the question above, the answer is quite varied, but the number of respondents who answers no was the most numerous by the number of 49 respondents or around 38% of the total respondents. And 4 other answers chosen but with less percentage. Information through VMS was not affected at this location because drivers are not guided toward an alternative route. So for the drivers who do not know the alternative route will not use the alternative route.

The next question is about the level of VMS benefits for road users can be seen from picture 6 below.

![Figure 6. The benefits degree of VMS for road users](image)

The answer to this question is very positive, namely respondents said that the VMS useful for road users with a total of respondents who chose reached 64 respondents or about 49%, while 59 respondents or 45% of the total respondents vote is very useful and the rest answered quite useful with the percentage reach 5%.

**Traffic survey result**

Implementation of traffic counting survey done by automated methods through tools Video Image Processor (VIP), which is mounted in such a way in accordance with the existing provisions. From the results of a survey the number of vehicles through the toll lanes PASUPATI FLY OVER in the direction to Gasibu units of time per vehicle type acquired. The length of time the implementation of a traffic survey is 5 x 6 hours, which is divided into two time intervals survey, i.e., morning and evening, 06: 00-09: 00 and 15: 00 to 18: 00. Time Interval measurement is done with the intent to get the peak hour traffic volume in the morning and the afternoon. Enumeration of traffic survey's activities carried out on 2 (two) stage activity that is, without using vms information vms and uses information by variable message sign (vms). Survey before use VMS
done on 8th -12th March 2013 and survey conducted on the 17th to 21st July 2013.
On each stages of survey enumeration the volume of traffic’s peak hours on every day obtained, as can be seen on a table 1.

**Table 1.** Traffic volume data before and after information given

<table>
<thead>
<tr>
<th>Before information given</th>
<th>After Information given</th>
</tr>
</thead>
<tbody>
<tr>
<td>Day</td>
<td>Date</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>Friday</td>
<td>8 March 2013</td>
</tr>
<tr>
<td>Saturday</td>
<td>9 March 2013</td>
</tr>
<tr>
<td>Sunday</td>
<td>10 March 2013</td>
</tr>
<tr>
<td>Monday</td>
<td>11 March 2013</td>
</tr>
<tr>
<td>Tuesday</td>
<td>12 March 2013</td>
</tr>
</tbody>
</table>

![Traffic volume Pasupati flyover directions to Gasibu before and after the information submitted through the VMS](image)

**Figure 1.** the comparison of the volume of traffic when before and after information given

With the passage of traffic conditions information through the medium of a Variable Message Sign (VMS) leads to a decrease in the volume of traffic through internode PASUPATI Flyover at peak hours, it
will be directly proportional to the possibility of traffic jams on the roads. It can be understood by comparing the graphs of traffic volume before and after the information is given in Figure 7, the average decline in traffic volume as a result of the passage of traffic information system through the VMS for 5 (five) day surveys is 20%.

CONCLUSION

Based on the results of the data traffic volume survey, traffic condition information system through the VMS quite successful in reducing the volume of traffic at peak hours, this can be evidenced by the decrease in peak hour traffic volume on the Road PASUPATI Flyover after information via VMS activated. The other thing strengthened these successes are the result of questionnaire data from where the road users answered that reinforces the need for VMS and VMS information influence for the existence of road users.

SUGGESTION

Larger-scale trials is needed, so that the information can be delivered not only about traffic condition information in one section alone but the traffic condition information on surrounding streets. It is necessary to know the effects of complete information on VMS for road users and expected this data can be an input for the Government in taking policy against traffic jam in Indonesia.

REFERENCE

2 http://ekbis.sindonews.com/read/694810/34/infrastruktur-jalan-masih-tertinggal
3 Hanafiah, Disi, (2012). Laporan akhir Model fisik Video Image Processing (VIP) dan Variable Message Signs (VMS), Pusjatan
PAPER TITLE
(90 Characters Max)
Analyzing Toll Road Service Quality From a Road User Perspective (Case Study of Toll Roads in Java)

TRACK

AUTHOR (Capitalize Family Name)
Irv. Herry T. ZUNA M.Sc
Ph.D Candidate
Civil Engineering Department,
University of Indonesia
Indonesia

CO-AUTHORS (S)
(Capitalize Family Name)
Prof. DR. Ir. Sigit P. HADIWARDOYO
Professor
Laboratory of Structures and Materials, Research of Road and Railway,
University of Indonesia
Indonesia

DR. Ir. Hedy RAHADIAN M.Sc
Head of Regional Office
Indonesia

E-MAIL
(for correspondence)
Hatezet@gmail.com

KEYWORDS:
Toll roads, Performance indicators, Quality attributes, Users’ perspective, Servqual

ABSTRACT:
Many researchers believe that toll road performance will be measured using travel time indicator. Though, there are other services perceived by customer while they are using toll road. In some case, perspective of service quality between government, operator, and customer may not match. This paper will focus on proposing service quality attributes of toll road service based on user’s perspective, which influence their decision to use toll road and their satisfaction of using it. Using SERVQUAL approach and Importance Performance Analysis, author reveals 8 service attributes that greatly affect customers satisfaction, most of them related to reliability and responsiveness dimension.
ANALYZING TOLL ROAD SERVICE QUALITY FROM A ROAD USER PERSPECTIVE  
(CASE STUDY OF TOLL ROADS IN JAVA) 

Ir. Herry T. Zuna SE, M.Sc¹, Prof. DR. Ir. Sigit P. Hadibardoyo², DR. Ir. Hedi Rahadian M.Sc³  
¹,² University of Indonesia, ³Directorate General of Highway  
Email for Correspondent: hatezet@gmail.com

1 INTRODUCTION

Mobility plays important role in urban and rural economy. In this decade, it continues to grow as a vital element of national development, especially due to economic growth. To fulfill the demand of mobility, new road provision emerges as a solution, which includes toll road provision. The objective of toll road provision is to spread development to all regions in Indonesia. In this sense, efficiency of development is necessary to boost economic growth, not only for urban area, but also in rural area.

In Indonesia, toll road provision falls under the authority of national government. To maintain and accelerate the provision of infrastructure including toll road, government adopt public private partnership (PPP) concept in particular using build-operate-transfer (BOT) scheme, where private company is authorized to design, build and operate toll roads in accordance with the concession agreement and at the time agreed upon ownership of toll road assets will be handed over to the government (Levy, 1996). However, in some cases there are situation where lack of quality control by government, operator providing unsatisfactory service, as the result of less attention to users’ expectation. So, the perspective of service quality level between government, operator, and road user/customer may not match (Zeithaml et al, 1990).

Many researches focus on travel time benefit as the main factor that influences the decision to use toll road (Jou et al, 2012). However, there is a suggestion that travel time benefit, which is very dependent on traffic density, is not the only determining factor in people’s decision to use the toll road (Choocharukul et al, 2004). Highway users continue to use the toll road although there is no time saving benefit gained because traffic congestion also occurred on toll roads. It is estimated that there are other factors that affect decisions such highway service quality. The study develops toll road service quality concept in Indonesia based on road user perspective. A similar study based on quality of service is commonly found in other transport modes, such as rail, airline, public transport (Eboli & Mazzulla, 2008). This paper will focus on proposing service quality attributes influencing to the road user’s decision to use toll road and their satisfaction of using it.

2 LITERATURE REVIEW ON CUSTOMER SATISFACTION AND SERVICE QUALITY MODEL

Users generally have an initial perception of a products or services offered. The perception of an individual's initial response may be positive or negative standpoints. Associated with quality of service, fulfillment of initial perceptions are very important and influential to a value of quality of service. According Tjiptono (2005), the quality of service defines the dynamic state associated with products, services, human resources, processes and environments that meet or exceed expectations.

Quality of service is a collection of public perceptions in a form of service. Another definition, service quality is a form of attitude representing the overall evaluation of long-term (eg Cronin and Taylor, 1994). Service quality is measured based on the user’s perceptions and expectations of a service to shape the quality of service that can be identified by considering the interests and user satisfaction (Eboli & Mazzulla, 2008). Customer satisfaction is a response to consumer demand which is a consideration for a product or service in providing enjoyable consumption levels. While Kotler (1995) defines customer satisfaction as the level of one's feelings after comparing the performance or results that he felt compared to expectations.

Gronroos model was the early conceptualization of service quality which defined service quality by three elements, namely technical quality (outcome), functional quality (process of delivery the service), and Image (Gronroos, 1984). Later, this model was developed by several author to proposed their own service quality model (Rust & Oliver, 1994; Parasuraman et al, 1988; Dabholkar et al, 1996; Brady & Cronin, 2001). Even some other model have been proposed, SERVQUAL. Model proposed by Parasuraman et al (1985) is the model often used by authors to measure service quality. Parasuraman et al (1990) mentioned five dimensions of SERVQUAL, known as RATER:
• Reliability: ability of providers to perform and deliver the promised services accurately and reliably.
• Assurance: or guarantee of knowledge, skill, and ability of service providers for creation of a trust.
• Tangible or physical evidence: a manifestation of the existence of a service provider to the user, this can include physical facilities, infrastructure and equipment.
• Empathy: provision of individual care and attention to customers
• Responsiveness: The willingness to help customers and provide prompt service

Zethaml et al (1990) adding measurement of disconfirmation paradigm to complete this model, by measuring gap or difference between expected service and perceived service by the user. Disconfirmation paradigm stated that if the service perceived is less than expected, mean the satisfaction is not met, otherwise if the service perceived by the user equal or greater than expected, the satisfaction is met (Jain & Gupta, 2004). Furthermore, This theory indentifies five gaps, which is: (1) The gap between customer expectation – perception of management; (2) The gap between management perception-perception of service quality; (3) the gap between service quality specifications and actual service quality; (4) The gap between the way the ministry of external communication about this service; (5) The gap between expectation and perception of the services.

SERVQUAL model has widely used to measure service quality in service sector, e.g. education service (Akhlagi et al, 2012), banking service (Ariff et al, 2013; Yousapronpaiboon, 2014), and health care service (Purcarea et al, 2013). Moreover, this model was also used in several public transport research e.g. bus service, train service, airport and airline service, although some of them have modified the dimension of SERVQUAL (Pabendinskaite & Akstinaite, 2014; Mustafa et al, 2005; Aydin & Yildirim, 2012; Randheer et al, 2012). Other authors also tried to measure user satisfaction based on service quality, but defining different variables from RATER dimensions (Shabaan & Khalil, 2013; Maravuda & Bellamkonda, 2013; Eboli & Mazulla 2006, 2007, 2008; Geetika, 2010, Litman, 2008).

Road, including toll road, is considered as a (public) goods. Performance of toll road generally defined based on road provision (preservation), added with outcome factors such as economy, environment, traffic, etc. (Humplick & Peterson, 1994; Hartanto & Susilo, 2001). While some authors stated that the most important factor is travel time benefit and traffic condition (Senbil & Kitamura, 2004; Jay et al, 2011; Susilawati et al 2008; Sakai et al, 2011). These indicators generally can be diagnosed and measured by operator using specific tools and methods, with slight involvement of customers’ viewpoints. In addition, many researchers believe that travel time, besides physical preservation, is the mostly affecting to customer satisfaction rather than other factor.

In this paper, authors try to think oppositely, that toll road is a service. Customers’ activities while using toll road should also be defined as toll road service quality. Customers are involved and inseparable in the value-creating process in service sector as a co-creator role (Gronroos, 2011). Customers’ perception and expectation can be a consideration to operator to define level of service quality delivered.

3 RESEARCH METHODOLOGY

This study using primary data from study of BPJT (2013) which in this research, spesifically using study case of 11 toll roads in Java Island, Indonesia, consisting 4 urban toll roads, and 7 inter-urban toll roads, Two thousand questionnaires were distributed to toll road users, with different number of questionnaire for each toll roads, based on its traffic volume. The questionnaire asked respondents about quality of toll road service attributes, both expectation (importance level) and perceived (performance level). The evaluation of these level ranked 1 to 5 on Linskert Scale with “1 = not important/not satisfied, 2 = least important/least satisfied, 3 = fair importance/fair satisfied, 4 = important/satisfied, 5 = very important/very satisfied.”

Parasuraman et al (1988) SERVQUAL model determined attributes of service quality based on 5 dimensions. In this research, authors determined 16 attributes based on SERVQUAL approach. These attributes represent RATER Dimension.

Many approaches to measure service quality and satisfaction level, including the methods of Importance Performance Analysis (IPA). IPA is a method proposed by Martilla & James (1977) to measure performance of a product or service using users’ satisfaction level and expectation level. Furthermore, this tool not only measure performance, but also provides a strategy for service quality improvement. IPA form a matrix using grand mean of importance value grand mean of performance value which divided into four quadrant: (1) High importance – high performance, called “keep up the good works”, (2) High importance – low performance,
called “concentrated here”, (3) Low importance – low performance, called “low priority”, and (4) Low importance – high performance, called “possible overkill”.

Using Importance Performance Analysis matrix, this paper assess service quality, focusing on its importance value (quadrant 1 and quadrant 2), with additional information of its performance level. Service attributes categorized as an important attributes when the importance value is greater than importance value mean (standard level of performance).

\[
\bar{Y} = \frac{\sum_{i=1}^{n} Y_i}{n} \quad \text{and} \quad \bar{X} = \frac{\sum_{i=1}^{n} X_i}{n}
\]

\[
Y_i = \text{importance value of attribute } i \\
X_i = \text{performance value of attribute } i \\
n = \text{number of attributes}
\]

Fig. 1 Importance Performance Analysis, source: adapted from Martilla & James (1977)

4 RESULT AND DISCUSSION

From the questionnaire data, the trip characteristics of toll roads user are identified as it can be seen in table 1. From this table, can be seen that most of respondent use toll road for work/business based trip and because it is have travel time benefit compared to non-toll road. The paradigm that main service of toll road is to reduce travel time have been embedded by most traveler, which consistent with some authors who considered travel time benefit as main indicator to measure road performance, especially toll road.

<table>
<thead>
<tr>
<th>Types of vehicle</th>
<th>Total</th>
<th>Urban Toll Road</th>
<th>Inter Urban Toll Road</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passenger</td>
<td>82%</td>
<td>90%</td>
<td>79%</td>
</tr>
<tr>
<td>Freight</td>
<td>18%</td>
<td>10%</td>
<td>21%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Frequency</th>
<th>Total</th>
<th>Urban Toll Road</th>
<th>Inter Urban Toll Road</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 times a week or more</td>
<td>41%</td>
<td>71%</td>
<td>38%</td>
</tr>
<tr>
<td>2-3 times a week</td>
<td>32%</td>
<td>17%</td>
<td>31%</td>
</tr>
<tr>
<td>1 time a week or less</td>
<td>27%</td>
<td>12%</td>
<td>31%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Purpose</th>
<th>Total</th>
<th>Urban Toll Road</th>
<th>Inter Urban Toll Road</th>
</tr>
</thead>
<tbody>
<tr>
<td>Work/business</td>
<td>78%</td>
<td>90%</td>
<td>78%</td>
</tr>
<tr>
<td>Leisure</td>
<td>6%</td>
<td>3%</td>
<td>7%</td>
</tr>
<tr>
<td>Others</td>
<td>16%</td>
<td>7%</td>
<td>15%</td>
</tr>
<tr>
<td>Reason</td>
<td>Total</td>
<td>Urban Toll Road</td>
<td>Inter Urban Toll Road</td>
</tr>
<tr>
<td>---------------------</td>
<td>-------</td>
<td>-----------------</td>
<td>----------------------</td>
</tr>
<tr>
<td>Time benefit</td>
<td>89%</td>
<td>91%</td>
<td>90%</td>
</tr>
<tr>
<td>Safety benefit</td>
<td>2%</td>
<td>2%</td>
<td>2%</td>
</tr>
<tr>
<td>Comfort benefit</td>
<td>5%</td>
<td>4%</td>
<td>5%</td>
</tr>
<tr>
<td>Other reason</td>
<td>4%</td>
<td>3%</td>
<td>3%</td>
</tr>
</tbody>
</table>

source: data analysis (2014)

Table 2 shows the result of average importance value and performance value based on users’ perspective for each toll road attributes. However, users’ perception defined all toll road attributes to be an important attributes (no attributes with importance value lower than 4), with “no traffic congestion”, “riding safety”, and “smoothness of road surface” as the 3 highest importance value. However, customers feel all of services were delivered below their expectation level.

Table 2. Importance Performance Value of Toll Road in Java

<table>
<thead>
<tr>
<th>Dimension</th>
<th>No</th>
<th>Attributes</th>
<th>Importance Value</th>
<th>Performance Value</th>
<th>Gap</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reliability</td>
<td>1</td>
<td>No traffic congestion</td>
<td>4.70</td>
<td>3.10</td>
<td>-1.60</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Riding safety</td>
<td>4.68</td>
<td>3.26</td>
<td>-1.41</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Smoothness of road surface</td>
<td>4.64</td>
<td>2.93</td>
<td>-1.71</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Accuracy of information given</td>
<td>4.14</td>
<td>3.37</td>
<td>-0.77</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Security from crime</td>
<td>4.44</td>
<td>3.45</td>
<td>-0.99</td>
</tr>
<tr>
<td>Assurance</td>
<td>6</td>
<td>Toll gates operator services</td>
<td>4.33</td>
<td>3.43</td>
<td>-0.91</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>Friendly toll gates operator</td>
<td>4.26</td>
<td>3.47</td>
<td>-0.79</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>honest toll gates operator</td>
<td>4.30</td>
<td>3.58</td>
<td>-0.72</td>
</tr>
<tr>
<td>Tangible</td>
<td>9</td>
<td>Toll gates facilities</td>
<td>4.32</td>
<td>3.36</td>
<td>-0.96</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>Lighting</td>
<td>4.62</td>
<td>2.92</td>
<td>-1.70</td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>Traffic sign and information board</td>
<td>4.33</td>
<td>3.49</td>
<td>-0.83</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>Facilities of rest area</td>
<td>4.21</td>
<td>3.36</td>
<td>-0.85</td>
</tr>
<tr>
<td>Empathy</td>
<td>13</td>
<td>Call center service</td>
<td>4.07</td>
<td>3.28</td>
<td>-0.79</td>
</tr>
<tr>
<td>Responsiveness</td>
<td>14</td>
<td>fast response of emergency unit</td>
<td>4.44</td>
<td>3.21</td>
<td>-1.23</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>accident handling</td>
<td>4.55</td>
<td>3.30</td>
<td>-1.26</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>responsiveness in road preservation</td>
<td>4.49</td>
<td>3.25</td>
<td>-1.24</td>
</tr>
<tr>
<td>Grand Mean</td>
<td></td>
<td></td>
<td>4.41</td>
<td>3.30</td>
<td></td>
</tr>
</tbody>
</table>

source: data analysis (2014)

Based on table 2, IPA matrix was created with grand mean of importance value (4.41) as standard level of importance, and grand mean of performance value (3.30) as standard level of performance. Figure 2 shows that there are 8 attributes plotted in quadrant 1 and 2, which have importance value greater than the standard. The attributes which need to be prioritized, (1) no traffic congestion, (2) riding safety, (3) smoothness of road surface, (5) security from crime, (10) road lightings, (14) fast response of emergency unit, (15) accident handling, and (16) road preservation. In addition, quadrant 1 only consist of one attributes, namely (5) security from crime, while rest of them have performance value below the standard (quadrant 2). It means, most of them are have less acceptable performance, and need higher concern from toll road operator to improve its level of service.
Figure 2. Importance-Performance Analysis Total Respondent

Figure 3 illustrates comparison of service quality attributes between urban toll road and inter urban toll road. Even both of them have different pattern of importance and performance coordinate, the list of important attributes are the same.

Table 3. Importance Performance Value of Urban and Inter Urban Toll Road

<table>
<thead>
<tr>
<th>Dimension</th>
<th>No</th>
<th>Attributes</th>
<th>Urban Toll Road</th>
<th>Inter Urban Toll Road</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Imp Value</td>
<td>Perf Value</td>
</tr>
<tr>
<td>Reliability</td>
<td>1</td>
<td>No traffic congestion</td>
<td>4.59</td>
<td>2.86</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Riding safety</td>
<td>4.59</td>
<td>3.27</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Smoothness of road surface</td>
<td>4.55</td>
<td>3.19</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Accuracy of information given</td>
<td>4.12</td>
<td>3.33</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Security from crime</td>
<td>4.39</td>
<td>3.52</td>
</tr>
<tr>
<td>Assurance</td>
<td>6</td>
<td>Toll gates operator services</td>
<td>4.27</td>
<td>3.41</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>Friendly toll gates operator</td>
<td>4.17</td>
<td>3.41</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>Honest toll gates operator</td>
<td>4.22</td>
<td>3.50</td>
</tr>
<tr>
<td>Tangible</td>
<td>9</td>
<td>Toll gates facilities</td>
<td>4.26</td>
<td>3.40</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>Lighting</td>
<td>4.59</td>
<td>3.18</td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>Traffic sign and information board</td>
<td>4.34</td>
<td>3.45</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>Facilities of rest area</td>
<td>4.06</td>
<td>3.32</td>
</tr>
<tr>
<td>Empathy</td>
<td>13</td>
<td>Call center service</td>
<td>4.07</td>
<td>3.27</td>
</tr>
<tr>
<td>Responsiveness</td>
<td>14</td>
<td>Fast response of emergency unit</td>
<td>4.41</td>
<td>3.25</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>Accident handling</td>
<td>4.48</td>
<td>3.27</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>Responsiveness in road preservation</td>
<td>4.53</td>
<td>3.29</td>
</tr>
</tbody>
</table>

|                |    |                                      | 4.35       | 3.31       | 4.44       | 3.30       |

Source: data analysis (2014)
4 CONCLUSION

Based on IPA method, operators can define which attributes should be prioritized to maintain user satisfaction. Toll road users have high expectation on the service provided by operator. However, for efficiency purpose, operator should focused on the most important attributes at the first place, namely: absent of traffic congestion, road safety, smoothness of road surface, security from criminal actions, road lightings, responsiveness of emergency unit, accident handling, and road preservation. It can be said that users are more concerned about services perceived while they are driving (reliability of toll road) and how the operator response for emergency situation (responsiveness of toll road operator). In addition, there are no difference between urban toll road and inter urban toll road. While the study more concerned on importance value of each service attributes, it also found that service level of toll road attributes was delivered with unacceptable performance.

While the function of this method is to define what attributes should be prioritized, this method rely too heavily on road user’s perception, which produced bias on its value. As it stated before, the value of importance level stated by users for each attributes are very high, and it affect the standard of performance level, resulting important attributes categorized as unimportant attributes. Instead of define attributes as important and unimportant, this methods can categorized attributes as priority and secondary, while the secondary group still need to get attention from the operator, but after priority standard have been met. To conclude, this tool can help operator to understand the needs of customers and invite them to be part of service creation process.

5 REFERENCE


Jou, R. C., Chiou, Y. C., Chen, K., Tan H. I. (2012) Freeway Drivers’ Willingness to Pay For a Distance Based Toll Rate. Transportation Research Part A 46:549-559


**PAPER TITLE**
Laboratory investigation of asphalt binder with and without crumb rubber modifier

**TRACK**

**AUTHOR**
| Saleh ALOTAIBI | Lecturer | King Abdul-Aziz University | Saudi Arabia |

**CO-AUTHOR(S)**

**E-MAIL**
esann28@hotmail.com

**KEYWORDS:**
Asphalt binder, temperature susceptibility, modification, crumb rubber and stiffness

**ABSTRACT:**
Countries around the world receive asphalt binder from different sources. The same asphalt binder grades or sources behave differently in the same environmental condition. This behaviour can be attributed to the diversity of sources. Although all these sources of binders have to pass characterisation tests, some of these sources are quite sensitive to temperature variations. This research aims to study the effects of the Crumb Rubber modification (CR) on the temperature susceptibility of the asphalt binders using traditional and Superpave techniques. Two asphalt binder grades were investigated: 60/70 was from New Zealand source and 80/100 was imported. The two binder sources were evaluated in the laboratory under three different conditions: unaged binder, short-term and long-term ageing. Furthermore, the response of the addition of 10% CR modifier was investigated under these various conditions. The results of this research clearly showed that the addition of the CR modifier led to reduced temperature susceptibility of the asphalt binder and improved resistance to thermal cracking at low temperatures.
Laboratory investigation of asphalt binder with and without crumb rubber modifier

Eng. Saleh Alotaibi

King Abdul-Aziz University, Civil Engineering Department, Rabigh Branch, Jeddah, KSA
esamm28@hotmail.com

1 INTRODUCTION

Asphalt binder ageing

It is widely acknowledged that the rheological properties of the asphalt binder have a direct influence on pavement performance. Once the Hot Mix Asphalt (HMA) productions and pavement applications take place, the rheological properties of the asphalt binder start to experience a change in their nature; consequently, asphalt binder is expected to have a critical condition over time (Robert et al. 1996).

The principal elements of asphalt molecules are carbon and hydrogen (Asphalt Institute 1994). Asphalt binder has a complex chemical composition that varies in response to the environmental conditions. Oxidation and physical hardening are the major factors that contribute to a change in the asphalt binder properties, which is clearly attributed to age hardening over the service life of the pavement, which consequently affects its behaviour (Bahia and Anderson 1995). Asphalt binder becomes brittle with age due to oxidation. Moreover, when asphalt pavement is exposed to a low temperature for a prolonged time, shrinkage and hardening occurs. Once asphalt binder has experienced hardening, the pavement structure is more likely to show cracking (Asphalt Institute 1994).

Asphalt binder temperature susceptibility

At a high temperature, asphalt binder behaves like a viscous liquid. Moreover, it is characterised as a Newtonian fluid that has a linear relationship between the force of resistance and its relative velocity. When the temperature drops or the pavement rapidly undergoes applied loads, the asphalt binder similarly acts as an elastic solid. Low temperatures cause the asphalt binder to become brittle; in addition, thermal cracking occurs when there is a low temperature and excessive load. Asphalt binder is deemed as a visco-elastic and thermoplastic material; it behaves completely differently when its temperature changes. As it is heated, it becomes fluid which would easily coat the aggregate; after cooling, it behaves like glue and holds the aggregate together (Asphalt Institute 1994). Temperature susceptibility is considered an important property of the asphalt binder. It can be defined as the rate of change of binder consistency as the temperature changes. Asphalt binder that is highly susceptible to temperature change could result in tender mix at high temperature because of the very low viscosity; similarly, very high viscosity could lead to thermal cracking at very low temperature (Roberts et al. 1996).

2 SCOPE OF RESEARCH

The main objective of this research project was to study the consequence of the Crumb Rubber modification (CR) on the temperature susceptibility of the asphalt binders using traditional and Superpave techniques. Two asphalt binder grades secured from two different sources were investigated: 60/70 and 80/100 imported; in addition, each asphalt binder source was modified with 10% (by weight of the asphalt binder) CR modifier. These asphalt binder samples were tested through three different conditions in order to simulate the real service life: unaged, rolling thin film oven test short-term aged and long-term aged. Unaged asphalt binder is subjected to transporting, manufacturing processes in its early life stages. Short-term ageing simulates the ageing during mixing, lay down and compaction of the pavement service life and the long-term ageing imitates the pavement performance over 7 to 10 years of service time or longer. Figure 1 shows a flow chart of the experimental design.
3 MATERIALS

The asphalt binders were provided by Fulton Hogan Limited, which brought them from Shell New Zealand Limited. Two asphalt binder sources were provided: 60/70 and 80/100 (imported), named in this research as asphalt binders A, B respectively. The same sources were modified with 10% of CR. The CR modified asphalt binders were prepared by adding 10% of crumb rubber passing number 40 mesh size, and 5% of aromatic oil Tudasen 65, and 80% of asphalt binder.

\[ PI = \frac{20 - 500A}{1 + 50A} \]  

\[ A = \frac{\log \text{Pen at } T_1 - \log \text{Pen at } T_2}{T_1 - T_2} \]  

\[ PVN = \frac{L - X}{L - M} (-1.5) \]  

Where, \( X \) is the logarithm of viscosity in centistokes measured at 135°C; \( L \) is the logarithm of viscosity at 135°C for a PVN of 0.0; and \( M \) is the logarithm of viscosity at 135°C for a PVN of -1.5. 

\( L = \log V = 4.25800 - 0.79670 \log P \)  

\( M = \log V = 3.46289 - 0.61094 \log P \)

4 METHODOLOGY

Figure 1 shows the testing programme that was followed. Each binder source with and without CR modifier was subjected to a number of laboratory tests through three stages: unaged, short-term and long-term aged binder. Penetration measurements were undertaken only for the unaged asphalt binder; however, viscosity measurements were conducted on unaged asphalt binders as well as artificially aged asphalt binders; BBR parameters were only measured for the artificially aged asphalt binders.

The hardening of the asphalt binder starts to take place during the manufacture and construction of HMA pavement. This type of aging is called short-term ageing because it occurs at the beginning of the pavement’s service life. The Rolling Thin-Film Oven Test (RTFOT) imitates short-term aging that the asphalt binder experiences in its early stage of life during manufacturing and placement (Robert et al. 1996). In order to simulate the long-term ageing that the HMA pavement is expected to experience during its service life, RTFOT was utilized for 24 hours at 125°C.

Traditionally there are three indexes that can be used to determine temperature susceptibility, viscosity temperature susceptibility (VTS), penetration viscosity number (PVN) and penetration index (PI). In the Superpave characterisation methods, the bending beam rheometer parameters, namely, creep stiffness and m-value are used to characterise the low temperature behaviour of the asphalt binder. Figure 2 displays the three apparatuses utilised to measures the temperature susceptibility parameters.

**Viscosity measurements**

All the unmodified binders and the 10% CR modified asphalt binder sources artificially experienced short-term ageing in the RTFOT for 85 minutes at 163°C according to (ASTM D 2872-97). The Rolling Thin Film Oven Test equipment was also used to simulate the long-term ageing, but the RTFOT kept ageing the asphalt binders for 24 hours at 125°C. The viscosity was measured at different temperatures before and after short-term ageing as well as after long-term ageing, for determining the asphalt binder temperature susceptibility. The viscosity of the different sources of binders was measured by using the Brookfield rotational viscometer (RV). Table 1 and Table 2 summarise the viscosity results at different temperature for the two different sources for different ageing and modification conditions.
Penetration measurements

In order to find the Penetration Index (PI) and Penetration-Viscosity Number (PVN), the penetration test was conducted at three different temperatures (room temperature, 25°C and 30°C) for both the unmodified asphalt binders and the 10% CR modified asphalt binder according to the (ASTM D5-06) standard. While the PI is only based on the penetration value at different temperatures, the PVN is based on both viscosity at 135°C and penetration at 25°C. This variety of testing temperatures is due to the specification of asphalt binder paving (Roberts et al. 1996). Those indexes are considered to examine the temperature susceptibility of the asphalt binder. PI and PVN were calculated using Equation 1 and Equation 2. The smaller the value of PI and PVN, the higher the temperature susceptibility of asphalt binder (Roberts et al. 1996).

![Diagram](image)

Figure 1. Experimental design.
Table 1. Summary of the viscosity versus temperature results and BBR’s parameters of the 0% CR modified binder sources

<table>
<thead>
<tr>
<th>Property</th>
<th>Temperature</th>
<th>Source A</th>
<th>Source B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Viscosity, Pa.s @</td>
<td>100 °C</td>
<td>5.569</td>
<td>3.2496</td>
</tr>
<tr>
<td></td>
<td>115 °C</td>
<td>1.96033</td>
<td>1.12883</td>
</tr>
<tr>
<td></td>
<td>135 °C</td>
<td>0.6563</td>
<td>0.5083</td>
</tr>
<tr>
<td></td>
<td>140 °C</td>
<td>0.52293</td>
<td>0.3625</td>
</tr>
<tr>
<td>Short-term aged binder (RTFO) residue (163°C)</td>
<td>Viscosity, Pa.s @</td>
<td>100 °C</td>
<td>8.87533</td>
</tr>
<tr>
<td></td>
<td>115 °C</td>
<td>2.49733</td>
<td>1.41467</td>
</tr>
<tr>
<td></td>
<td>135 °C</td>
<td>0.89167</td>
<td>0.49167</td>
</tr>
<tr>
<td></td>
<td>140 °C</td>
<td>0.6417</td>
<td>0.377</td>
</tr>
<tr>
<td></td>
<td>Stiffness (60 Sec). MPa (-20°C)</td>
<td>296</td>
<td>641</td>
</tr>
<tr>
<td></td>
<td>m-Value (60 Sec) (-20°C)</td>
<td>0.310</td>
<td>0.219</td>
</tr>
<tr>
<td>Long-term aged binder (RTFO) residue (125°C)</td>
<td>Viscosity, Pa.s @</td>
<td>135 °C</td>
<td>1.675</td>
</tr>
<tr>
<td></td>
<td>140 °C</td>
<td>1.251</td>
<td>0.545</td>
</tr>
<tr>
<td></td>
<td>150 °C</td>
<td>0.723</td>
<td>0.342</td>
</tr>
<tr>
<td></td>
<td>163 °C</td>
<td>0.404</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>Stiffness (60 Sec). MPa (-20°C)</td>
<td>353</td>
<td>551</td>
</tr>
<tr>
<td></td>
<td>m-Value (60 Sec) (-20°C)</td>
<td>0.272</td>
<td>0.253</td>
</tr>
</tbody>
</table>

Table 2: Summary of the viscosity versus temperature results and BBR’s parameters of the 10% CRMA binder sources

<table>
<thead>
<tr>
<th>Property</th>
<th>Temperature</th>
<th>Source A</th>
<th>Temperature</th>
<th>Source B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Viscosity, Pa.s@</td>
<td>135 °C</td>
<td>2.26</td>
<td>115 °C</td>
<td>4.10</td>
</tr>
<tr>
<td></td>
<td>140 °C</td>
<td>1.74</td>
<td>135 °C</td>
<td>1.54</td>
</tr>
<tr>
<td></td>
<td>145 °C</td>
<td>1.48</td>
<td>140 °C</td>
<td>1.25</td>
</tr>
<tr>
<td>Short-term aged binder (RTFO) residue (163°C)</td>
<td>Viscosity, Pa.s@</td>
<td>135 °C</td>
<td>4.49</td>
<td>115 °C</td>
</tr>
<tr>
<td></td>
<td>140 °C</td>
<td>3.50</td>
<td>135 °C</td>
<td>2.32</td>
</tr>
<tr>
<td></td>
<td>145 °C</td>
<td>2.75</td>
<td>140 °C</td>
<td>1.82</td>
</tr>
<tr>
<td></td>
<td>Stiffness (60 Sec). MPa (-20°C)</td>
<td>238</td>
<td>350</td>
<td></td>
</tr>
<tr>
<td></td>
<td>m-Value (60 Sec) (-20°C)</td>
<td>0.301</td>
<td>0.333</td>
<td></td>
</tr>
<tr>
<td>Long-term aged binder (RTFO) residue (125°C)</td>
<td>Viscosity, Pa.s@</td>
<td>135 °C</td>
<td>4.02</td>
<td>135 °C</td>
</tr>
<tr>
<td></td>
<td>140 °C</td>
<td>3.51</td>
<td>140 °C</td>
<td>1.68</td>
</tr>
<tr>
<td></td>
<td>145 °C</td>
<td>2.71</td>
<td>150 °C</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td>Stiffness (60 Sec). MPa (-20°C)</td>
<td>200</td>
<td>277</td>
<td></td>
</tr>
<tr>
<td></td>
<td>m-Value (60 Sec) (-20°C)</td>
<td>0.289</td>
<td>0.308</td>
<td></td>
</tr>
</tbody>
</table>

**BBR parameters measurements**

In accordance with (ASTM D 6648-01), the BBR test was conducted on the RTFOT residues at -20°C for unmodified asphalt binders after short-term ageing as well as after long-term ageing, three duplicate beam samples (125x6.35x12.7mm) were tested after conditioning them in the methanol bath at the desired low temperature. Similarly, the same procedure was followed at -20°C for the 10% CR modified binders.
\[ VTS = \frac{\log \log \text{viscosity at } T_2 - \log \log \text{viscosity at } T_1}{\log T_1 - \log T_2} \]

Where, T1 and T2 are the binder temperatures in Kelvin

Figure 1: (a) Penetrometer, (b) Rotational Viscometer (VR), (c) Bending Beam Rheometer (BBR)

5 RESULTS AND DISCUSSION

Viscosity-temperature susceptibility index (VTS)

The Brookfield rotational viscometer was used to measure the viscosity of all unmodified asphalt binders and the 10% CR modified asphalt binder sources at different temperatures in order to study how they can be distinguished by using VTS parameters. Table 3 indicates for the VTS values, there was no major difference after subjecting unmodified asphalt binders to short-term and long-term ageing. This means that the temperature susceptibility of all the unmodified asphalt binder sources had no changes before and after ageing.

The 10% CR modified asphalt binders, however, displayed an improvement in VTS for unaged asphalt binders. A higher VTS value demonstrates high temperature susceptibility asphalt binder (Roberts et al. 1996). Table 3 shows that 10% CR modified asphalt binder source B had the highest VTS. There were significant differences in their VTS. Regardless of the asphalt binder sources, the VTS of short-term aged 10% CR modified asphalt binder and long-term aged 10% CR modified asphalt binder did not show significant differences among the asphalt binder sources. Generally, the asphalt binder sources showed significant improvement in terms of temperature susceptibility when they were modified with 10% crumb rubber by weight. Equation 3 was used to calculate the VTS.

<table>
<thead>
<tr>
<th>Binder sources</th>
<th>VTS of Source A</th>
<th>VTS of Source B</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0 % CR</td>
<td>10 % CR</td>
</tr>
<tr>
<td>Unaging</td>
<td>3.2</td>
<td>2.4</td>
</tr>
<tr>
<td>Short-term aging</td>
<td>3.3</td>
<td>2.5</td>
</tr>
<tr>
<td>Long-term aging</td>
<td>3.3</td>
<td>2</td>
</tr>
</tbody>
</table>
Penetration parameters

Penetration tests were conducted at three different temperatures, including room temperature, in order to study the behaviour of the different asphalt binder sources. Both unaged unmodified asphalt binders and unaged 10% CR modified asphalt binders were assessed. Temperature susceptibility parameters, PI and PVN, were calculated. Table 4 summarises all the parameters for both the unmodified asphalt binder sources and the 10% CR modified asphalt binder sources.

Table 4: Penetration Index (PI) and Penetration-Viscosity Number (PVN) with and without crumb rubber modification

<table>
<thead>
<tr>
<th>Binder sources</th>
<th>Source A</th>
<th>Source B</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0 % CR</td>
<td>10 % CR</td>
</tr>
<tr>
<td>PI</td>
<td>-0.88</td>
<td>0.98</td>
</tr>
<tr>
<td>PVN</td>
<td>-0.05</td>
<td>1.42</td>
</tr>
</tbody>
</table>

The asphalt binder that has a lower value of PI and PVN will have higher temperature susceptibility (Roberts et al. 1996). From this, the PI of the unmodified asphalt binder source B has the highest temperature susceptibility. Similarly, the PVN of the unmodified asphalt binders was presented showing a similar trend.

The temperature susceptibility of various asphalt binder sources has been found to be significantly affected by the CR modified asphalt binder (Lee et al. 2008). Table 4 shows that PI and PVN were remarkably improved after the modification with the crumb rubber. Both PVN and PI methods showed a different temperature susceptibility order, however, all binder sources showed some improvement of the temperature susceptibility after the modification with crumb rubber. Figure 3 illustrates the relationship between penetration and temperature for both unmodified binder sources and 10% CR modified asphalt binder sources; the higher the temperature the higher the penetration.

![Figure 3: Penetration-Temperature relationship: (a) Unaged unmodified asphalt binders (b) Unaged 10% CR modified asphalt binders](image)

Bending Beam Rheometer’s parameters (BBR)

The BBR test is one of the Superpave tests that is able to imitate temperature and loading conditions. This test was conducted on the RTFOT residue after artificial short-term and artificial long-term ageing. Creep stiffness and m-value parameters were measured for both modified and unmodified asphalt binder sources at -20°C. Table 1 and Table 2 summarise the m-values and creep stiffness values.
From Figure 4, it can be seen that the m-value of the short-term aged asphalt binder source A decreased after being modified with 10% crumb rubber, but the m-value of source B was improved. However, regardless of the Superpave specification limit, the m-value of all the long-term aged asphalt binder sources had a significant improvement with the CR modifier.

Regardless of the modification, Figure 5 indicates an increase in the creep stiffness after a longer time of ageing of the asphalt binder source A, while source B showed a significant reduction in creep stiffness as ageing time increased. Asphalt binder source A behaved differently compared with source B; this was also observed for m-value. This discrepancy might be because of the waxy content and chemical composition of the asphalt. The molecular structure of the asphalt binder is affected differently with either heating or cooling due to differences in size and the chemical bonding type among asphalt binder sources (Asphalt Institute 1994). Generally, irrespective of the asphalt binder sources, 10% CR modified asphalt binders had a significant improvement in the creep stiffness that showed more resistance to thermal cracking at low temperatures.

![Figure 4: Effect of asphalt binders with and without CR modifier on m-value](image)

![Figure 5: Effect of asphalt binders with and without CR modifier on creep stiffness](image)

*STA: Short-term ageing, LTA: Long-term ageing
6 CONCLUSION

The results of investigation of the thermal characteristics of the New Zealand and overseas asphalt binders, with and without CR modifier, resulted in the following:

- The addition of the CR modifier significantly influenced the viscosity of the asphalt binder. High CR modifier content demonstrated higher viscosity of the asphalt binder which reduced the temperature susceptibility of the asphalt binder. This was observed not only after short-term ageing, but also after long-term ageing.
- For unmodified asphalt binder, the increase in viscosity due to long-term ageing was more pronounced compared to short-term ageing. However, for CR modified asphalt binder sources, the viscosity was lower for long-term ageing compared with short-term ageing.
- Both m-value and creep stiffness was more influenced by the addition of crumb rubber showing good improvement.

7 REFERENCES


**PAPER TITLE**: Summary of Construction of Shorenji river work section on Yodogawa Sagan route of Hanshin Expressway

**AUTHOR**

<table>
<thead>
<tr>
<th>Capitalize Family Name</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Toshiro Nagasawa</td>
<td>—</td>
<td>Hanshin Expressway Company Limited</td>
<td>Japan</td>
</tr>
</tbody>
</table>

**Co-Author(s)**

<table>
<thead>
<tr>
<th>Capitalize Family Name</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
</table>

**E-MAIL** (for correspondence)
toshiro-n_hanshin6036x@hotmail.co.jp

**KEYWORDS:**
backfill
soil cohesion
N-value
soil improvement
PCB

**ABSTRACT:**

Yodogawa Sagan route is a part of “Osaka Metro Renaissance Loop Route” which was planned to disperse car flows of incoming to the city and through traffic and reduce traffic jams in Osaka city consequently.

The section of 5.7 KMs is called the first phase of Yodogawa Sagan route, connecting Hokko JCT in Wangan line of Hanshin Expressway route 5 with Ebie JCT in Kobe line of Hanshin Expressway Route 3, completed in May 25, 2013.

Shorenji River Work Section is 2.3km tunnel section going under the old Shorenji River after back filling, three relevant entities; Osaka prefectural government, Osaka municipal government and Hanshin Expressway Company Limited agreed and jointly conducted the backfilling and the construction of cut-and-cover tunnel for Yodogawa Sagan Route and the transformation of the surface into park and promenade.

The ground of tunnel work section in the river has soft ground properties, and it is backfilled after the tunnel structures are constructed. Therefore, allowable bearing power examination of the said ground was carried out. And the batholith improvement in the most part excavated range is carried out for gaining the necessary bearing power under the tunnel section.

This paper reports the summary of construction works of the cut-and-cover tunnel.
Summary of Construction of Shorenji River work section on Yodogawa Sagan route of Hanshin Expressway

Toshiro Nagasawa

1 Hanshin Expressway Company Limited, Chuo-Ku, Osaka, Japan
Email for correspondence: toshiro-nagasawa@hanshin-exp.ne.jp

1 INTRODUCTION

Hanshin Expressway is toll urban highway with total network length of 259.1km in Osaka and Kobe and the neighbouring areas. The feature of the expressway is that it mainly consists of radiated intercity routes connecting suburban areas and loop route in the centre of Osaka city. Hanshin Expressway supports the economy of the Kansai Area and is a base infrastructure of daily living. However, chronic traffic jams are noticeable inside Osaka city. That is caused by vehicles pass through the central Osaka which is called “through traffic”. In order to solve such situation the second outer loop route called “Osaka Metro Renaissance Loop Route” was planned. The plan is to disperse car flows of incoming to the city and through traffic and reduce traffic jams consequently. It will also contribute to the reduction of the amount of exhausted gas by the improvement of driving speed.

The 5.7km section of the second loop in the west is called the first phase of Yodogawa Sagan Route, connecting at Hokko JCT, with Wangan Route No. 5 and at Ebie JCT with Kobe Route No. 3. The route was completed in May 25, 2013. It enable to go directly from Osaka Bay areas to inner areas without passing through the city center, and congestion alleviation in the inner city can be expected. This section along with the planned second phase of Yodogawa Sagan route extending to east to Shin-Midosuji line is a part of Osaka Urban Ring Road. (Figure 1)

Figure 1. Hanshin Expressway in Osaka city

Figure 2. Position of Shorenji River work section
Shorenji River Work Section is a 2.3km tunnel section going under the old Shorenji River after backfilling (Figure 2). Originally Shorenji River was a branch river of Yodo River system and used for industrial water supply and boat transport route since the industrial development in circa 1900s. It served an important function in development in western Osaka City. However local communities demanded backfilling of the river due to the decline of ship transportation that started from around 1965, degradation of water quality and geographical separation of the local areas. Therefore three relevant entities; Osaka prefectural government, Osaka municipal government and Hanshin Expressway Company Limited agreed and jointly conducted the backfilling and the construction of cut-and-cover tunnel for Yodogawa Sagan Route and the transformation of the surface into park and promenade. (Figure 3)

This paper reports the summary of construction works of the cut-and-cover tunnel.

2 WORK SUMMARIES

Figure 4 shows the construction flow of backfilling and tunnel works. At first, steel pipes were casted in the center of the river to dry the right bank side. The right bank side was then reclaimed and consolidated with accumulated sludge in the riverbed. Secondly, the artificial river and sewage box were made to maintain the conventional river function on the right bank side. To prevent the emission of strong ammoniac odor which happened after the refilling and re-excavation of the right bank side, a different method was employed for the left bank. The excavated sludge was...
3 RIVER RECLAMATION

At first, steel pipes were installed in the river center to the longitudinal direction. Then, sludge in the riverbed was solidified by stirring a machine on the spot while injecting cement-type solidification materials. (Picture 1) The solidification of the sludge was carried out in a submergence state for preventing the odor dispersing. After the solidification was finished, water was drained off and outer layer of solidified sludge was improved in the same manner. The purpose of improving outer layer was to get traffficability that was the essential requisites for construction of the river and sewage box and the tunnel.

Shorenji river was one of affluent of Yodo river system, but following the facts of ground subsidence and numerous damages by high tide and flooding, the upstream part which was connected with Yodo River had been already backfilled before the project. Also water supply pipes for industrial use had been installed on the same spot. Furthermore, maintenance pumping for river water purification and environment conservation was performed from the tidal compartment of Yodo River with the speed of 22 m³/s. Therefore, 3.4km-long river box was re-installed before the backfilling and starting the tunnel work. It is because the conventional river functions of flood control and water supply should be maintained. On the other hand, it was planned to maintain the sewage box to preserve sewerage functions of the Shorenji River in the same manner. Because sewerage of areas along a river were combined system and water was expected to be released to the river beyond the processing capacity in case of a heavy rain, it was projected to build a supplemental trunk sewer route to meet the timing of new pumping station in the downstream. The project of this sewage box was carried out by Osaka Prefectural Government.

When the backfilled right bank side was re-excavated and its unevenness was corrected, strong ammoniac odor was started to be emitted. The causative agent of the bad smell was included in the bottom part of cement improvement soil. The management was difficult because the odor occurred whenever a new reclaimed surface was exposed even though the deodorant dispersion. Therefore, a different method should be considered for the left bank side from the viewpoint of environmental conservation of neighboring communities. After the considerations, the combined methods of dehydrating and solidifying the excavated sludge in a separated plant was employed. (Picture 2) After dredging the sludge of riverbed by the vacuum aspiration transfer method, and shipping it to the processing facilities through pipelines, the soil was spin-dried and solidified by a hyper-pressure filter system. Then the processed soil and the excavated ordinary soil from the tunnel work were used for backfilling.

4 SOIL IMPROVEMENT AND POLLUTED SLUDGE DISPOSAL

As a geological characteristic of the Western Osaka area as shown in Figure 5, AC1 layer which is soft alluvial clay layer of more than 10m in thickness is distributed almost horizontally right under the tunnel position. The lower part consists of alternated layers of the sand layer and the clay layer. Furthermore, the DG1 layer which is under high artesian pressure is distributed underneath. The tip of the temporary support wall is inserted into near the central part of the AC1 layer. Before the design of the cut-and-cover tunnel, more than 40 boring sampling and geological analysis were carried out along the tunnel route. Here, the soft ground AC1 layer that is important to the design of the tunnel is focused on, and characteristics of N-value and soil cohesion are determined. Those are particularly important parameters to design the temporary construction structure.

As shown in Figure 6, the average N-values of all data in the inner and outer river cross-section is about N=2 ~ 3 which indicates the entire area is very soft ground. In addition, the tunnel construction section in the river owns the tendency that the mean value is smaller than outer river section.

The soil cohesion value (c) has a big varied distribution pattern, and the variation gets bigger in the depth direction. In addition, there is a tendency where the soil cohesion value in the inner river section is small along with N-value. Based upon the aforementioned results, for the fixed geological parameters to use for the design of the tunnel, the
N-value is set to be mean value of all data of the inner river section and soil cohesion value is set by using the primary equation of regression by using all data.

The ground of tunnel work section in the river has soft ground properties, and it is backfilled after the tunnel structures are constructed. Therefore, allowable bearing power examination of the said ground was carried out. And the batholith improvement in the most part excavated range is carried out for gaining the necessary bearing power under the tunnel section. Regarding the choice of the batholith improvement method, Medium-depth mixing method is adopted in the place where the necessary machine can be installed on the ground without any height limit and obstacle. (Picture 3) And in the place where there is any special condition, jet grouting method is adopted.

![Figure 5. Geological feature longitudinal section in Shorenji river](image)

![Figure 6. Distribution of the depth direction of Soil Cohesion and N-value](image)

![Picture 3. Medium-depth mixing method](image)

5 TUNNEL CONSTRUCTION

Figure 7 shows the construction flows of the main structure of the tunnel. Firstly, the temporary support walls are constructed. Secondly, piles to support temporary construction cat walk are driven followed by the construction cat walk. There are existing structures on site such as river bank wall or river and sewage box in the proximity to the position of tunnel structures. In addition, there are six old bridges crossing the old river including wide artery of National Highway Route 43. In the place where the tunnel interferes with the foundations of the bridges, the tunnel is constructed while removing the footing and pile foundation and supporting the bridge pier and upper structure with temporary substructure (Picture 4). It is expected that the remaining foundations of several bridges have a big influence on the excavation because of the soft ground and, adjacent to the cofferdam for tunnel construction, that there is other cofferdams built for the construction of the river and sewage box. Therefore elasto-plastic analysis and FEM analysis are conducted to confirm that those displacement values are less than the allowable value before the commencement of tunnel construction. In addition, the displacement of cofferdam and the adjacent structures are measured during the construction to monitor that measured values are all in the allowable level. In case a big displacement is observed, an appropriate countermeasure is taken.

The excavation is carried out from the temporary stage while carefully sorting soil so that toxic soil contains mercury, lead, PCB is separated from good quality soil suitable for backfilling. Since the tunnel box uses massive amount of premix concrete, based on the thermal analysis in advance, low-heat-generating Portland cement is chosen for sidewalk and slabs members. The harmful thermal cracks were not identified in the investigation after the construction.
図-3.2.1 正蓮寺橋計測管理断面図

図-3.2.1 正蓮寺橋計測管理断面図

図-7. Construction flows of the main structure of the tunnel

図-8. Measurement management section (at the point of Shorenji Bridge)
6 SETTING OF TEMPORARY RIVER WALL

Figure 9 shows the separation point of Shorenji river and Rokkennya river, that had been connected with the Yodo river, and the expressway is planned to plunge into the right bank in the same place. It had been necessary to carry out the construction of the expressway maintaining the function of the flood control. Therefore it was necessary to construct a temporary river wall which substitutes the existing river wall when the construction of the expressway interfered with it. The setting is as follows.

[1] Making temporary water channel
A temporary channel wall was constructed in the river to make the place maintaining the function of the flood control basin. In the figure 9, the orange line shows the existing river wall, which interfere the tunnel construction.

[2] Constructing temporary river wall & Removing existing river wall
A temporary river wall was constructed as shown in the figure. After the temporary river wall had been built, the existing river wall was removed. The expressway tunnel was built afterwards (green part).

![Air photograph](image)
[3] Relocating temporary river wall
The temporary river wall was relocated to construct the expressway tunnel just the same place [2]. The new temporary river wall was constructed on the slab of already constructed tunnel. (Picture 5) In addition, the temporary wall of the RC structure was constructed inside of the already constructed tunnel to maintain the river wall function.

[4] Finish constructing the tunnel
After the relocation of the temporary river wall, the rest of the expressway tunnel was constructed. The design of the temporary river wall was based in "special bank design manual (plan October, 2006)" of Osaka prefecture.

7 TREATING WASTE SOIL

In 1999, the PCB of the density more than "provisional removal standards of the quality of bottom" was detected in the Shorenji River in the sludge of the riverbed. In addition, total mercury of the density more than the standards was detected from an illegally dumped thing.

- The range and the density of the sludge in the riverbed of the PCB
  Range: about 50m (width), about 800m (length), deeper than 1-3m from the riverbed of the left bank side
  The highest density: 960 mg/kg (the standard value: 10 mg/kg or less)

- Illegally dumped industry thing
  Range: about 4m (width), about 40m (length)
  Total mercury density: up to 27 mg/kg (the standard value: 25 mg/kg or less)

In addition, not only PCB and total mercury but also dioxin derived from PCB was detected from the sludge in the riverbed. The illegally dumped thing is regarded as the thing about 30-40 years ago, the PCB more than standards is not to be touched from outside the river. So the river manager confirmed that there was not the influence on neighboring inhabitants promptly.

Figure 10. Position to containment soil contains PCB
As emergency procedure, the river manager surrounded the illegal dumping point by sheet piles and covered the surface with concrete immediately.

On the basis of the following things, the permanent measure were considered.

- The ground did not have the contact with the person directly.
- A pollution range was clear.
- The polluted sludge was confined in the underground part and was in a stable state.
- There were impermeable layers (the clay layer) no less than the thickness of 10m under the polluted sludge, and there was not groundwater contamination.
- The solidification technology and the containment technology at the original position were secured.

Based upon it, the method to confine the area of the river concerned and covering up with soil was estimated as the safest and most effective measures method. Therefore, after the polluted sludge was spin-dried, and it was solidified and improved, and it was bottled up in the river. (Figure 10) As for the truck, the location was managed using GPS so that the truck might not go to other places without permission. (Figure 11)

Most of the soil contains mercury and lead. It was forbidden taking the soil including those heavy metal outside the river area. All the soil had been used it for the backfill of the tunnel in river areas. In addition, cover earth was laid not to let human beings to touch the soil dilectry.

8 CONCLUSION

In Shorenji River work section, the cut-and-cover tunnel was constructed under the backfilled Shorenji River. Much conference and adjustment on a river manager and a sewer manager were put and the expressway tunnel construction was pushed forward while the existing river function maintained. Local communities demanded backfilling of the river due to the decline of ship transportation, degradation of water quality and geographical separation of the local areas. Therefore three relevant entities; Osaka prefectural government, Osaka city government and Hanshin Expressway Company Limited agreed and jointly conducted the backfilling, the construction of cut-and-cover tunnel for Yodogawa Sagan Route and the transformation of the surface into park and promenade.

It was expected that the remaining foundations of several bridges have a big influence on the excavation because of the soft ground, and some proximity structures for tunnel construction. Therefore elasto-plastic analysis and FEM analysis were conducted to confirm that those displacement values were less than the allowable value before the commencement of tunnel construction. In addition, the displacements of proximity structures were measured during the construction to monitor that measured values were all in the allowable level.

In the future, the appropriate management of the soil including pollutants such as the PCB is important, and it is necessary to watch it so that a pollutant does not begin to leak. Particularly, it should be careful about management not to pollute groundwater and not to make any leaking into the tunnel of the expressway in future.
**PAPER Title**
Cost-Effective Safety Treatment of Culverts and Bridges on Low-Volume Rural Roads

**TRACK**

<table>
<thead>
<tr>
<th>AUTHOR</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Karla A. LECHTENBERG</td>
<td>Research Associate Engineer</td>
<td>Midwest Roadside Safety Facility, University of Nebraska-Lincoln</td>
<td>USA</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CO-AUTHOR</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cody S. STOLLE</td>
<td>Post-Doctoral Research Associate</td>
<td>Midwest Roadside Safety Facility, University of Nebraska-Lincoln</td>
<td>USA</td>
</tr>
<tr>
<td>Ronald K. FALLER</td>
<td>Research Associate Professor and Director</td>
<td>Midwest Roadside Safety Facility, University of Nebraska-Lincoln</td>
<td>USA</td>
</tr>
<tr>
<td>Kevin D. SCHRUM</td>
<td>Research Engineer</td>
<td>University of Alabama at Birmingham</td>
<td>USA</td>
</tr>
</tbody>
</table>

**E-MAIL**
kpolidka2@unl.edu

**KEYWORDS:**
Low-Volume Roads, Safety Improvement, Benefit-to-Cost Analysis, Cost-Effective, Culverts, and Bridges

**ABSTRACT:**
A benefit-to-cost analysis was performed to investigate the efficacy of safety treatment alternatives for culverts and bridges located on roadways with traffic volumes less than 500 vehicles per day and posted speed limits of 55 mph or greater. A field survey was performed to develop a parametric model, which included roadway geometries as well as culvert/bridge geometry and frequency. Approximately 1500 culvert/bridge configurations were analyzed with ten different traffic volumes ranging from 50 to 500 ADT. Several safety treatment methods were considered: (1) “Do Nothing”, which represented the baseline condition; (2) removal of existing, non-crashworthy rail; (3) installation of a crashworthy guardrail or bridge rail system; and (4) installation of traversable grates for culvert structures.

Benefit-to-cost ratios were calculated and used to make recommendations based on the length and vertical drop-off for culverts/bridges and lateral offset of the culvert/bridge away from roadway edge. For a majority of culverts, removal of the existing, non-crashworthy rail was considered the most cost-beneficial option if it existed. However, for rural bridge rails on low-volume roads, the best option was often to allow existing bridge rails to remain in place due to removal and replacement costs. Roadway engineers are encouraged to use these guidelines as a foundation for making future safety improvements with consideration of a site-specific analysis.
COST-EFFECTIVE SAFETY TREATMENT OF CULVERTS AND BRIDGES ON LOW-VOLUME RURAL ROADS

Ms. Karla A. Lechtenberg
Dr. Cody S. Stolle
Dr. Ronald K. Faller
Dr. Kevin D. Schrum

1Midwest Roadside Safety Facility, University of Nebraska-Lincoln, Lincoln, Nebraska, USA
2University of Alabama at Birmingham, Birmingham, Alabama, USA
Email for correspondence: kpolivka2@unl.edu

1 INTRODUCTION

Culverts and bridges are one of the most common roadside fixed objects located along low-volume roads. Safety treatments for culverts, bridges, and any other drainage features (e.g., drainage channels, pipes, and drop inlets) located along low-volume roads have traditionally consisted of field-constructed barriers or hazard indicators, such as delineators and object markers. Historical barriers constructed on top of culvert headwalls and on bridges vary in design and construction, and include wood post-and-beam designs, angle-iron systems, and concrete post-and-beam configurations. However, many of the barrier designs are not crashworthy and pose a greater risk to errant vehicles than the culvert or bridge opening which the barrier was intended to shield. Smaller rails, constructed from wood or small angle-iron sections, are too weak to prevent vehicles from passing over or penetrating through the barriers, and thus are essentially no safer than omitting the culvert or bridge rail completely. The potential for those rails to penetrate the vehicle compartment may make them even more severe than the unprotected culvert or bridge opening and vertical drop off.

The American Association of State Highway Transportation Officials (AASHTO) Roadside Design Guide (RDG) (AASHTO 2006) primarily provides roadside design guidance for moderate- to high-speed, high-volume highways and roadways, while providing limited guidance for low-volume, local roads and streets. In general, much of the latter guidance was extrapolated from higher-speed and higher-volume design guidelines. As a result, the guidelines for most rural, local roads are only loosely based on actual research results and may be impractical for local road applications due to right-of-way and financial constraints. There are few safety treatments along low-volume roadways due to the perception that few cost-effective treatments are available for a reasonable severity reduction.

The AASHTO Guidelines for Geometric Design of Very Low Volume Local Roads (AASHTO 2001) gives cursory coverage to roadside safety for roads with ADTs less than 400 vpd. In essence, improvements are only recommended where a documentable accident history exists. The very low traffic volumes produce sparsely populated accident histories. As a result, a single serious accident can dramatically affect the apparent need for safety treatment.

2 RESEARCH OBJECTIVE

The research objective for this study was to develop cost-effective recommendations for the safety treatment of culverts and bridges commonly found along roadways with traffic volumes less than 500 vpd and with posted speed limits of 55 mph (88.5 km/h) or greater by utilizing benefit-to-cost analyses. The primary research goal was to identify the most cost-effective safety treatment alternative based on roadway geometry, culvert and bridge geometry, and traffic characteristics.

3 FIELD INVESTIGATION

Locations

A limited field survey was undertaken to determine the common characteristics of culverts and bridges placed along very low-volume roadways. The field study was conducted in Marshall County, Kansas.
The Kansas Department of Transportation (KDOT) and Marshall County officials identified two continuous stretches of rural roadways, which represented typical very low-volume roadways and conditions. One stretch was 8 miles (12.9 km) long, and the other segment was 13 miles (20.9 km) long. These non-paved (i.e., gravel) roadways had traffic volumes less than 500 vpd and posted speed limits of 55 mph (88.5 km/h). The field survey was limited in order to make the research study manageable and was not intended to be all-inclusive.

**Observations**

**Culvert Profiles**

Culverts observed along the investigated low-volume roads varied in sized, shape, and type of culvert opening and typically spanned creeks or streams. Common culverts observed included: (1) small pipe set in the center of a rock wall and (2) box culvert with a concrete post-and-rail system. Culvert lengths (measured parallel to the roadway) varied from 9.3 to 20.5 ft (2.8 to 6.2 m). Culverts were typically less than 20 ft (6.1 m) long. Lateral widths of the culverts (measured perpendicular to the roadway) ranged from 8.25 to 15 in. (210 to 381 mm). Culvert heights ranged from 39.5 to 70 in. (1,003 to 1,778 mm).

The observed culverts typically consisted of a concrete headwall extending 3 to 8 in. (76 to 203 mm) above the road surface. Concrete post-and-beam structures were constructed on these headwalls. Concrete posts were commonly spaced approximately 36 in. (914 mm) on center, with one or two rectangular beams between the posts. The size of the concrete posts varied and were typically 6 to 9 in. (152 to 229 mm) wide (parallel to the road), 9 to 15 in. (229 to 381 mm) deep (perpendicular to the road), and 20 to 36 in. (508 to 914 mm) tall. Very little reinforcement was observed between the post and rail connection.

Some culverts included barriers consisting of 2-in. x-2 in. (51-mm x 51-mm) wood beams mounted on 2-in. x 2-in. by 36-in. (51-mm x 51-mm by 914-mm) wood posts, spaced approximately 4 ft (1.2 m) on center. Other treatments included various sizes of angle-iron and channel sections, typically less than 3 in. (76 mm) in width. Examples of culverts observed during the field study are shown in Figure 1. Collected field data included culvert length, width, and height, lateral culvert offset away from roadway edge, roadway width, shoulder width, and traveled-way width. Additional details on the collected field data can be found in the referenced research report (Schrum et al. 2012).

![Culvert - Concrete Post and Rail](image1.png) ![Culvert – Wood Post and Rail](image2.png)

Figure 1 Typical Culverts – (a) Concrete Post and Rail and (b) Wood Post and Rail

**Bridge Profiles**

Bridges observed along the investigated low-volume roads appeared to have been in place for a significant period of time. In addition, the safety treatments typically did not appear to satisfy the safety performance criteria of the National Cooperative Highway Research Program (NCHRP) Report No. 350.
The various bridge railing types included: (1) an angle-post and rail design; (2) a variation of W-beam guardrail; and (3) a through-truss bridge with steel sections for beams and posts.

The angle-post bridge rail design utilized 3-in. x 3-in. by 20-ft (76-mm x 76-mm by 6.1-m) long steel angle rails supported by 3-in. x 3-in. by 6-ft (76-mm x 76-mm by 1.8-m) long steel angle posts, spaced 3 ft (0.91 m) on center. The W-beam bridge rail system consisted of rectangular concrete sections with 6-in. wide x 6-in. long x 12-in. tall (152-mm x 152-mm by 305-mm) wooden blockouts. Round head bolts with 2-in. long x 1-in. wide x 12-gauge thick (51-mm x 25-mm x 2.67-mm) steel plate washers attached the W-beam guardrail to the concrete posts. Resurfacing was observed to have reduced the top rail mounting height for the W-beam guardrail. A reduction in top rail mounting height has been shown to result in reduced performance of W-beam guardrail systems (Wright & Robertson 1976). In addition, thick grass and shrub growth in front of the approach guardrail prevented effective delineation of the bridge. Examples of bridges observed during the field study are shown in Figure 2. Collected field data included bridge length, width, and height, bridge rail height from the roadway, lateral bridge offset away from roadway edge, roadway width, and traveled-way width. Additional details on the collected field data can be found in the referenced research report (Schrum et al. 2012).

![Bridge - Angle Post and Rail](image1)
![Bridge – Concrete Posts with W-beam](image2)

Figure 2 Typical Bridge Railings – (a) Angle Post and Rail and (b) Concrete Posts with W-beam

4 RSAP ANALYSIS

Overview

This study was based on a parametric analysis of the characteristics found during a real-world site survey. Several roadway geometry, culvert geometry, and bridge geometry parameters were incorporated into the Roadside Safety Analysis Program (RSAPv2) models. Once the baseline models with relevant parameters were developed, safety treatment options were identified. RSAP was used to analyze each scenario under a variety of roadway and traffic characteristics. The results of the RSAP runs were used to determine recommendations for the treatment of existing culvert and bridge rail treatments.

RSAP Functional Class Coding Error

A version of RSAP (version 2003.04.01), which utilizes an interface to allow users to develop the necessary data files for the executable file to simulate the scenarios, was used during this research study. Unfortunately, the RSAPv2 FORTRAN code contained a logical, but fixable error. When specifying a local highway, the user interface creates a model using a more severe speed and angle distribution associated with freeways. Left unmodified, this error provides results with accidents having abnormally-high severity indices. Therefore and to account for this coding error, the data file created by the user interface was modified to allow the functional class code to use the speed and angle distribution of local highways instead of freeways.
A detailed explanation of this coding error as well as the associated resolution can be found in the referenced research report (Schrum et al 2012).

Road Geometry and Modeling

Vertical grades and horizontal curves were common on low-volume roadways and can influence the number of accidents that occur on these roadways. Areas on hills or at crests would likely correlate with a more stringent safety treatment of roadside hazards than those found on straight, level road sections. Historical analyses of vertical curvature have shown that encroachments and crash frequency are greater on curved road sections and roads with grades as compared to straight road sections (Wright & Robertson 1976). Thus, an analysis of straight, level roads is believed to be conservative. In other words, any recommendations for treating culverts and bridges found on straight roads with level terrain should also be applied to roadways with vertical grades and horizontal curves due to a higher likelihood of serious impacts.

The roadway was modeled as a rural local road with two lanes of travel and an undivided median. It was modeled as a straight section with no vertical grade and a length of 1,000 ft (304.8 m). The roadside culvert and bridge were located at approximately the center of the section or 500 ft (152.4 m). The modeled lane width was 12 ft (3.7 m), because the lane widths found during the field survey ranged from 10 ft (3.0 m) to 15 ft (4.6 m) with most 12 ft (3.7 m) wide. Shoulder width was set to 2 ft (0.6 m); since, it has been demonstrated to have little effect on the results (Sicking et al. 2009). The nominal percent of trucks was set to two percent, and the speed limit was 55 mph (89 km/h). The traffic growth factor, which is the anticipated annual traffic growth rate expressed as a percent, was zero. This means that one would not expect the amount of traffic to increase in subsequent years. The encroachment rate adjustment factor was left at the default value of 1.0 and is intended to be used for special situation when the encroachment rate is expected to differ significantly from the average. If locations have a higher than average encroachment history, a value greater than 1.0 is used. Conversely, if a value less than 1.0 is used, it means the location has a less than average encroachment history. However, encroachment frequencies vary widely on low-volume roadways.

Culvert and Bridge Geometry and Modeling

Besides modeling the road geometry, the variables necessary to develop the culvert and bridge model had to be determined. The modeling parameters for the culvert and bridge are shown in Figure 3. The modeled values for each parameter are shown in Table 1 and Table 2 for culverts and bridges, respectively.

![Roadway Diagram](image)

Figure 3  Representative Culvert and Bridge Parameters and Locations.
Table 1 Culvert Modeling Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Offset, F</td>
<td>ft (m)</td>
<td>2, 3, 4, 5, 6 (0.6, 0.9, 1.2, 1.5, 1.8)</td>
</tr>
<tr>
<td>Drop-Off, H</td>
<td>ft (m)</td>
<td>1.3, 7, 13 (0.3, 1, 2, 4)</td>
</tr>
<tr>
<td>Culvert Length, L</td>
<td>ft (m)</td>
<td>4, 6, 8, 10, 12 (1.2, 1.8, 2.4, 3.0, 3.7)</td>
</tr>
<tr>
<td>ADT</td>
<td>vpd</td>
<td>50, 100, 150, 200, 250, 300, 350, 400, 450, 500</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Slope Profile, SR</th>
<th>Long Slope</th>
<th>Ditch</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Flat, 8H:1V, 6H:1V, 4H:1V, 3H:1V, 2H:1V, 1.5H:1V</td>
<td>1.5H:1V and -1.5H:1V, 2H:1V and -2H:1V</td>
</tr>
</tbody>
</table>

Table 2 Bridge Modeling Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Offset, F</td>
<td>ft (m)</td>
<td>0, 3, 5 (0, 0.9, 1.5)</td>
</tr>
<tr>
<td>Drop-Off, H</td>
<td>ft (m)</td>
<td>7, 13, 20 (2, 4, 6)</td>
</tr>
<tr>
<td>Bridge Length</td>
<td>ft (m)</td>
<td>25, 50, 100, 150 (7.6, 15.2, 30.5, 45.7)</td>
</tr>
<tr>
<td>ADT</td>
<td>vpd</td>
<td>50, 100, 150, 200, 250, 300, 350, 400, 450, 500</td>
</tr>
<tr>
<td>Slope Profile</td>
<td></td>
<td>1.5H:1V</td>
</tr>
</tbody>
</table>

Culvert Profiles

Culverts were modeled using the dimensions observed in the field investigation. Five culvert lengths and four culvert heights (drop-offs) were chosen for the analysis. It was necessary to simulate the culvert with a vertical drop-off behind the culvert headwall, as was prevalent in the field investigation. Culverts and vertical foreslope drops were modeled by specifying a lateral offset from the edge of the travel way to the obstacle. Instead of using RSAP’s default culvert or vertical foreslope models, intersecting slopes were chosen. Within the intersecting slopes category, vertical drop-offs were used to model the ground or creek behind the culvert. Steep foreslopes were also common on low-volume roads. Therefore, leading up to the modeled culvert, a 2H:1V or 1.5H:1V foreslope was configured. A backslope was included beyond the foreslope to replicate a common ditch configuration found along low-volume roads. The selected predefined culvert depths for drop-offs were 1, 3, 7, and 13 ft, (0.3, 1, 2, and 3 m) deep, which were the smallest four drop heights available in RSAP and also represented the culvert depths observed in the field investigation.

A critical aspect of the culvert modeling was the concrete posts attached to the top of the concrete headwall. A conservative approximation was used to model the concrete support posts by using RSAP’s predefined rigid rectangular object. The representative post size was selected to be a 1.5-ft wide by 3-ft tall (0.5-m by 1-m) fixed object, which was larger than most of the posts observed during the field investigation. As a result, the severities were based on a larger object which would overestimate the post severity by a small amount. This conservative approach would place a small emphasis on using more crashworthy designs or doing away with the existing configurations.

Bridge Profiles

Two types of bridge railings were modeled in RSAP using the dimensions observed in the field investigation. Bridge type 1 consisted of an angle iron railing with a height of 31½ in. (0.8 m) above the bridge deck. In RSAP, this bridge rail was model as a Test Level 1 (TL-1) bridge rail with blunt ends. Basic dimensions measured from one of the bridges with an angle iron railing included a length of 69.75 ft (21.26 m) and a depth of 15.17 ft (4.62 m), as measured from the top of the bridge deck to the water in the creek.

Bridge type 2 consisted of concrete posts attached to the bridge deck with W-beam guardrail mounted on the face of the concrete posts. The top of the rail was typically 22 in. (0.56 m) above the road, which was less than the minimum required W-beam guardrail height (AASHTO 2006). Therefore, this system was also modeled as a TL-1 bridge rail with blunt ends in RSAPv2. The posts were 8 to 10 in. (0.20 to 0.25 m) wide and spaced on 6.25 ft (1.9 m) centers.
Lateral Offset

Since a very low-volume roadway has a clear zone of 12 to 14 ft (3.7 to 4.3 m) for 6H:1V or flatter slopes based on recommendations provided in the RDG (AASHTO 2006), county and local governments are not responsible for treatment of hazards outside of this window. The RSAP module was based on data derived from accidents on roadways with typical lane widths of 12 ft (3.7 m) or greater. Therefore, for roadways with widths greater than 24 ft (8.3 m), the road geometry was approximated by holding the lane width constant at 12 ft (4.2 m) and offsetting the culvert and slopes. For culverts, the offset values of 2, 3, 4, 5, and 6 ft (0.6, 0.9, 1.2, 1.5, and 1.8 m) were determined based on the actual road width. For bridges, the offset values of 0, 3, and 5 ft (0, 0.9, and 1.5 m) were determined based on the actual road width.

Side Slope

An important consideration when modeling culverts and bridges was the definition of side slopes, commonly referred to as fill slopes or foreslopes. The severity of the side slope varied based on slope rates, which may increase rollover propensity. No safety treatments were considered to shield the vehicle from the side slopes. A constant slope width was evaluated to ensure that the severity of each slope was based on roadside geometry. Therefore, only the depth of the vertical drop was adjusted within each segment length.

For culverts, side slopes may be steeper than 1.5H:1V and may have widths greater than 50 ft (15.2 m). A total of seven different slopes were considered, including 1.5H:1V, 2H:1V, 3H:1V, 4H:1V, 6H:1V, and 8H:1V as well as flat terrain. The slope widths for all slopes was set to 52 ft (15.9 m) to capture the longest option but to minimize the excessive distance behind the clear zone. Furthermore, RSAP used a cubic polynomial to determine the probability of lateral extent. The coefficients used in the polynomial provided positive probabilities at lateral offsets less than 18 ft (5.5 m). Beyond the 18-ft (5.5-m) offset, the calculated probability was negative, and the program adjusted that probability to zero. Therefore, even though the slopes were extended out to 52 ft (15.9 m), only the first 18 ft (5.5 m) were useful in the analysis. This range still fell within the clear zone of the roadway, which was generally between 12 and 18 ft (3.7 and 5.5 m).

For bridges, the procedure to determine the side slopes for culverts was still valid. However, only one slope of 1.5H:1V was considered.

Safety Treatment Options

Do Nothing

Alternatives were compared to a baseline option known as the “Do Nothing” alternative. This option allowed the existing rail to remain in place in its original configuration.

Remove Existing Rails

For both culverts and bridges, one treatment option was removal of the existing railing system that does not meet crashworthy standards. The cost of the removal was dependent upon what type of system was present. Further, the removal option was necessary if any other treatment option, such as installation of a crashworthy system, was considered. Therefore, the removal of existing rail was treated as the new baseline when analyzing the remaining treatment options.

For culverts, KDOT officials indicated that the likely method of removing the existing system would be by using a ball hammer on a crane, knocking the posts off of the culvert, and disposing of the debris at a disposal location. The cost of removing one post from the headwall was estimated to be approximately $1,000 which includes travel to and from the site and renting a dump truck and a ball hammer attachment for a crane bucket. For each additional post, crew and equipment use costs were estimated at $100. The $1,000 charge essentially represents a mobilization cost, and the post removal costs were dependent on the number of posts at a given culvert. It is assumed that both sides of the culvert railing and headwall would be removed in one trip. For example, post-removal costs for both sides of the roadway were estimated at $1600 for culvert lengths less than 10 ft (3.0 m) (i.e., 3 posts per side or a total of 6 posts for the culvert) and $1800 for
culverts with spans greater than or equal to 10 ft (3.0 m) (i.e., 4 posts per side or a total of 8 posts for the culvert).

For bridges, cost estimates for removing existing bridge rail were again provided by KDOT. Costs were broken into two areas: (1) removal of the existing bridge rail and (2) traffic control, mobilization, and contingency. Removal of the existing bridge rail was determined to be $20 per linear foot ($65.62 per linear meter) for steel-angle iron rail. To remove the W-beam guardrail with concrete posts, the cost along the bridge length was $20 per linear foot ($65.62 per linear meter) and $5 per linear foot ($16.40 per linear meter) for the approach and terminal section. Traffic control and mobilization were estimated to be 10 percent and 7.5 percent of the total cost, respectively. The traffic control cost was not to exceed $2,000. Contingency was included as 15 percent of the total cost, which covers anything that might not be covered in the other costs.

*Install Longitudinal Barrier*

Another treatment option was the installation of a crashworthy guardrail system to prevent vehicles from dropping off the edge of the culvert or bridge by capturing or redirecting vehicles. For this treatment option, it was necessary to remove the existing rail from the culvert headwall or bridge deck. As a result, the cost estimation for installing guardrail included the cost of removing the existing rail. Traffic control and mobilization were estimated to be 10 percent and 7.5 percent of the total cost, respectively. The traffic control cost was not to exceed $2,000. Contingency was included as 15 percent of the total cost, which covers anything that might not be covered in the other costs.

For culverts, a test level 2 (TL-2) guardrail and end terminal system was modeled for the longitudinal barrier. Since many culverts are low-fill box culverts with simple spans, this may allow the use of a long-span W-beam guardrail system with an unsupported length placed across the culvert, which would be the most economical alternative for shielding culverts that are less than 25 ft (7.62 m) wide. The guardrail installation was positioned with the front face of the rail 2 ft (0.61 m) in front of the culvert drop-off location, which was based on current recommendations for unsupported W-beam and long-span Midwest Guardrail System (MGS) (Bielenberg et al. 2007a, Albuquerque et al. 2009, Bielenberg et al. 2007b). W-Beam guardrail costs from the State Highway Agencies in Colorado, Kansas, Montana, Nebraska, Oregon, and Tennessee were averaged to obtain cost estimates for the RSAP analysis. The average cost was found to be $18.16 per linear foot ($59.58 per linear meter). A second cost for W-beam guardrail installations of $45 per linear foot ($147.64 per linear meter) was obtained from KDOT. Since this cost was significantly higher than the other averaged states, an analysis of the same scenarios was considered using these costs. A minimum guardrail length of 137.5 ft (41.91 m) was recommended based on estimated guardrail runout lengths developed by Wolford and Sicking (Sicking & Wolford 1996, Wolford & Sicking 1996).

For bridges, three components were necessary for an adequate bridge rail system: (1) a bridge rail; (2) an approach transition section; and (3) end terminals. Recall, the costs included the removal of the existing bridge rail, traffic control and mobilization, contingency, and the installation of a bridge rail, approach transition section, and end terminals. The costs for removal, traffic control, mobilization, and contingency were stated in the previous section. Cost estimates for this alternative were also provided by KDOT. The cost for installing an adequate retrofit bridge rail was $100 per linear foot ($328.08 per linear meter). The approach transition section cost was $50 per linear foot ($164.04 per linear meter). The terminal cost was estimated at $2,100 per 37.5-ft (11.4-m) long terminal. The terminals were modeled as 12.5 ft (3.8 m); and the cost of the extra 25 ft (7.6 m) on each end was subtracted from the cost of the approach transition section.

*Install Culvert Grate – Culverts Only*

Another treatment option, which only applied to culverts, was the installation of a culvert grate onto the existing side slopes. With this option, it was assumed that the culvert was in good condition and was capable of handling the loads imparted to it by the culvert grate during impact events. KDOT supplied estimated costs for culvert grate construction and equipment based on steel weight, concrete volume, and any necessary reinforcement, which included labor costs associated with each material. The cost to remove an
existing historical system was included in the total cost of the grate installation. Mobilization and extra equipment costs were estimated to be approximately 30 percent of the direct cost associated with culvert grate installation. Many of the culverts that were evaluated in the benefit-to-cost analyses were sized differently than culverts constructed with grates in Kansas. Thus, the culvert grate costs provided by KDOT were divided into four groups: (1) culvert grates installed with flared wingwalls and constructed on 3H:1V slopes; (2) culvert grates installed with straight wingwalls and located on 3H:1V slopes; (3) culvert grates installed with straight wingwalls and located on 6H:1V slopes; and (4) culvert grates installed on pre-existing wingwalls located on 3H:1V slopes. The cost associated with installing only a culvert grate is significantly less than when wingwall construction is required and formed the basis for breaking culvert grate costs down into four groups. Additional details on the costs to install culvert grates can be found in the referenced research report (Schrum 2012).

Delineation

Another treatment option could include installation of delineation devices to warn motorists of hazards located near the roadway. Based on various surveys from state departments of transportation, delineators are credited with a 30 percent and 15 percent reduction in roadside departures and run-off-road crashes on curves and straight road sections, respectively (AASHTO 2006, Labra & Mak 1980, Agent et al. 1996). As with longitudinal barriers, delineation may reduce the number of run-off-road excursions that occur on low-volume roadways as they indicated a hazard is located beyond the traveled way, but they do not shield the culvert or bridge rail. Due to difficulties of quantifying the benefits of delineation, this treatment option was not considered in the RSAP analyses. Thus, an in-service performance evaluation of the delineation alternative could be used to investigate its effectiveness in a variety of low-volume roadway conditions.

5 SIMULATION RESULTS

Analyses were performed to evaluate the cost-effectiveness of various safety treatment alternatives for culverts and bridges found adjacent to rural, low-volume roads and within the clear zone. The analysis was conducted based on observations and site data obtained during the field study of road geometry and culvert and bridge geometries. Approximately 1,500 culvert and bridge scenarios were analyzed. Each culvert and bridge configuration combination was analyzed with ten different traffic volumes ranging from 50 to 500 ADT in increments of 50. Culvert and bridge safety treatment recommendations were generated from the simulated scenarios by interpolating between the scenario results. These recommendations for culverts and bridges were made for B/C ratios of 2 and 4, as shown in Table 3 through Table 6, respectively. Additional details on the simulated scenarios can be found in the referenced research report (Schrum et al. 2012).

Culvert Simulation Results

For a benefit-to-cost ratio of 2, in combination with culvert drop heights of 1 to 3 ft (0.3 to 0.9 m) as well as fill slopes 3H:1V or shallower, removal of the existing system was cost-effective at an ADT as low as 50 vpd for all road widths. For a benefit-to-cost ratio of 4.0, “Do Nothing” was recommended only for ADT less than 100 vpd and was not recommended for an ADT greater than 250 vpd on most roadways. Additionally, the recommendation to install guardrail was generally restricted to roads with fill slope of 1.5H:1V or steeper. As road widths increased, the recommendation to install guardrail decreased.

For benefit-to-cost ratios of 2.0, long-span W-beam guardrail was recommended for traffic volumes as low as 100 vpd. The tendency to recommend this treatment option increased as the drop height and culvert length increased. However, the recommendation tendency decreased as the approaching slope flattened. Road width also affected the recommendations, such that, as the width increased, traffic ranges recommended for guardrail installation increased from 100 vpd on 30-ft (9.1-m) roads to 150 vpd on 36-ft (11.0-m) roads with drop heights of 2 ft (0.6 m). As drop height increased, traffic volume ranges expanded for the guardrail option.

Culvert grates were also considered. This option was not viable on drop heights of less than 3 ft (0.9 m). It was only sparsely cost-effective for drop heights of less than 8 ft (2.4 m), but it became cost-effective
for drop heights greater than 8 ft (2.4 m) and for culvert lengths less than 10 ft (3.0 m). If a wingwall had to be installed, and if recommendations were made to support this alternative, they were only made in the upper traffic volume ranges, such as 450 vpd or more. As road widths increased and slopes flattened, the propensity for using culvert grates was reduced.

Culvert grates were typically recommended for culverts less than 8 ft (2.4 m) long and more than 4 ft (1.2 m) deep and with foreslopes of 3H:1V and 4H:1V. Some 10-ft (3.0-m) long culverts and some culverts with 2 ft (0.6 m) depths were also recommended for culvert grate treatment. Installation of guardrail was typically recommended for ADT greater than 100 vpd for roads with a side slope of 1.5H:1V and for ADTs greater than 250 vpd for roads with side slopes of 2H:1V.

For benefit-to-cost ratios of 4.0, long-span W-beam traffic volume recommendations increased to 300 vpd on 30-ft (9.1-m) wide roads and drop heights of 4 ft (1.2 m). As the width increased to 36 ft (11.0 m), that volume increased to 400 vpd. As before, and as drop height increased, the propensity to recommend culvert grate installation increased but only when drop heights exceeded 8 ft (2.4 m). Additionally, culvert grate installation which required the construction of wingwalls was only recommended for one scenario: road widths between 30 ft and 32 ft (9.1 m and 9.8 m), drop heights greater than 8 ft (2.4 m), culvert lengths less than 4 ft (1.2 m), and slopes of 3H:1V.

On culverts with 4H:1V fill slopes, the most common recommendation was the installation of culvert grates. Culverts with steeper slopes were more often treated with guardrail to prevent the vehicle from traversing the non-recoverable slopes. However, culvert grates were recommended for 3H:1V slopes that had drop heights greater than 8 ft (2.4 m), even though AASHTO classifies 3H:1V slopes as non-recoverable slopes, which means that vehicles are not expected to return to the roadway after a departure (AASHTO 2006). Additionally, rollovers are more likely to occur on 3H:1V slopes than 4H:1V or 6H:1V slopes, thus indicating a lower risk to errant motorists by placing culvert grates on 4H:1V slopes than on 3H:1V slopes. Therefore, culvert grates were recommended for 3H:1V fill slopes or flatter.

**Bridge Simulation Results**

For a benefit-to-cost ratio of 4.0, the bridge analyses indicated that “Do Nothing” was preferred. The alternative to remove the existing rail always had a negative benefit-to-cost ratio, thus indicating that the accident cost without the rail was higher. Note that this finding is strongly correlated to the fact that neither the steel nor concrete system alternatives incorporated exceptionally strong posts. This finding is also due to the existing rail system being modeled as a TL-1 system. As a result, the existing barrier systems that were evaluated proved to have some beneficial effect. The findings may have been different if more rigid posts or end sections had been incorporated or if the capacity of the existing rail system is less than a TL-1 system.

For a benefit-to-cost ratio of 2.0, it became more beneficial to install an approved crashworthy bridge rail as the drop height increased. At a drop height of 7 ft (2.1 m), the minimum ADT was 450 vpd for installing an approved bridge rail. At a 13-ft (4.0-m) drop height, the minimum ADT was 400 vpd for installing an approved bridge rail. Finally, at a drop height of 20 ft (6.1 m), the minimum ADT was 350 vpd for installing an approved bridge rail. These minimum ADT’s were the same for either an existing angle iron rail or an existing W-beam rail. However, the results indicated a wider range of bridge lengths and lateral offsets over which it was economical to replace the angle iron with an approved bridge rail.

**6 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS**

Treatments for various culvert and bridge configurations were considered and analyzed to determine the most cost-effective treatment for such structures. Treatment options included doing nothing, removing existing non-crashworthy features from the culvert headwall and bridge deck, installing long-span guardrail across the culvert and bridge rail on the bridge, and installing a culvert grate for culverts only. Culvert treatment recommendations considered either installing only the culvert grate on existing wingwalls or constructing wingwalls before installing the culvert grate.
Benefit-to-cost ratios were generated through the use of RSAPv2 and were used to determine the most cost-effective safety treatment for culverts and bridges in various configurations of length, depth, and side slope. Recommendations indicated that if non-crashworthy features existed on the culvert headwall, it was often cost-effective to remove those features. However, for bridges that have non-crashworthy features, it was often recommended to leave the non-crashworthy rail in place since an accident without rail was more costly than an accident with the existing rail. Further, if a bridge under consideration is significantly different than that modeled, a site-specific benefit-to-cost analysis may be needed.

Culvert grate recommendations were strongly dependent on the culvert dimensions. For longer culverts, the benefit of the culvert grate did not increase as rapidly as the cost of installation. Culvert length did not have a significant effect on treatment recommendations, except for grates. The existence of wingwalls was found to have a significant effect on the benefit-to-cost ratios for the installation of culvert grates.

Delineation may prove to be effective for inattentive or impaired drivers by alerting motorists of an obstacle. This may reduce the number and speed of impacts due to heightened awareness. However, many crashes are the result of avoidance maneuvers, traffic violations, and mechanical failures. These crashes are typically not sensitive to delineation. Furthermore, delineation will not result in a reduction in the severity of an impact. Instead, additional investigation may be desired to evaluate how delineation may affect speed distribution and encroachments on very low-volume roadways where obstacle treatment guidelines indicated that removal of the existing non-crashworthy system or installation of a new crashworthy system was not a cost-effective solution. Individual analysis may be needed based on clearly-defined and quantifiable safety improvements for delineation.
Table 3 Culvert Treatment Recommendations for Low-Volume, Rural Roadways, B/C=2

<table>
<thead>
<tr>
<th>Slope Rate SR</th>
<th>Road Width W (ft)</th>
<th>Drop Height H (ft)</th>
<th>Culvert Length L (ft)</th>
<th>Do Nothing</th>
<th>Remove Posts</th>
<th>Install Guardrail</th>
<th>Culvert Grate if wingwalls otherwise Remove Posts</th>
<th>Culvert Grate if wingwalls otherwise Install Guardrail</th>
<th>Culvert Grate</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 30</td>
<td>&lt; 2</td>
<td></td>
<td>&lt; 5</td>
<td>0-49</td>
<td>50-99</td>
<td>100-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>≥ 5</td>
<td>0-49</td>
<td>50-99</td>
<td>100-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2 - 3.9</td>
<td></td>
<td>&lt; 7</td>
<td>0-49</td>
<td>50-99</td>
<td>100-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>≥ 9</td>
<td>0-99</td>
<td>50-99</td>
<td>100-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4 - 7.9</td>
<td></td>
<td>all</td>
<td>0-49</td>
<td>50-99</td>
<td>100-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>≥ 8</td>
<td>0-99</td>
<td>50-249</td>
<td>250-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>≥ 7</td>
<td>0-99</td>
<td>100-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5:1</td>
<td>&lt; 2</td>
<td></td>
<td>&lt; 7</td>
<td>0-49</td>
<td>50-199</td>
<td>200-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>30-31.9</td>
<td></td>
<td>≥ 9</td>
<td>0-99</td>
<td>100-149</td>
<td>150-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>32-35.9</td>
<td></td>
<td>≥ 5</td>
<td>0-99</td>
<td>100-149</td>
<td>150-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥8</td>
<td></td>
<td>≥ 7</td>
<td>0-99</td>
<td>100-149</td>
<td>150-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>≥36</td>
<td>&lt; 2</td>
<td></td>
<td>&lt; 7</td>
<td>0-99</td>
<td>100-149</td>
<td>150-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2 - 3.9</td>
<td></td>
<td>≥ 9</td>
<td>0-99</td>
<td>100-149</td>
<td>150-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4 - 7.9</td>
<td></td>
<td>≥ 7</td>
<td>0-99</td>
<td>100-149</td>
<td>150-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2:1</td>
<td>&lt; 2</td>
<td></td>
<td>&lt; 5</td>
<td>0-49</td>
<td>50-249</td>
<td>250-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>30-31.9</td>
<td></td>
<td>≥ 9</td>
<td>0-99</td>
<td>250-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4 - 7.9</td>
<td></td>
<td>≥ 7</td>
<td>0-99</td>
<td>250-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>32-35.9</td>
<td>&lt; 2</td>
<td></td>
<td>&lt; 5</td>
<td>0-49</td>
<td>50-349</td>
<td>350-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2 - 3.9</td>
<td></td>
<td>≥ 9</td>
<td>0-99</td>
<td>250-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4 - 7.9</td>
<td></td>
<td>≥ 7</td>
<td>0-99</td>
<td>250-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥8</td>
<td></td>
<td>&lt; 5</td>
<td>0-49</td>
<td>50-249</td>
<td>250-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4 - 7.9</td>
<td></td>
<td>≥ 7</td>
<td>0-49</td>
<td>50-249</td>
<td>250-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥8</td>
<td></td>
<td>&lt; 5</td>
<td>0-49</td>
<td>50-249</td>
<td>250-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4 - 7.9</td>
<td></td>
<td>≥ 7</td>
<td>0-49</td>
<td>50-249</td>
<td>250-500</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Table 3  Culvert Treatment Recommendations for Low-Volume, Rural Roadways, B/C=2 (continued)

<table>
<thead>
<tr>
<th>Slope Rate SR</th>
<th>Road Width W (ft)</th>
<th>Drop Height H (ft)</th>
<th>Culvert Length L (ft)</th>
<th>Do Nothing</th>
<th>Remove Posts</th>
<th>Install Guardrail</th>
<th>Culvert Grate if wingwalls otherwise Remove Posts</th>
<th>Culvert Grate if wingwalls otherwise Install Guardrail</th>
<th>Culvert Grate</th>
</tr>
</thead>
<tbody>
<tr>
<td>2:1</td>
<td>≥ 36</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 2</td>
<td>all</td>
<td>0-49</td>
<td>50-349</td>
<td>350-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2 - 3.9</td>
<td>all</td>
<td>0-49</td>
<td>50-349</td>
<td>350-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4 - 7.9</td>
<td>all</td>
<td>0-49</td>
<td>50-349</td>
<td>350-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 8</td>
<td>all</td>
<td>0-49</td>
<td>50-299</td>
<td>300-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 2</td>
<td>all</td>
<td>0-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2 - 3.9</td>
<td>&lt; 9</td>
<td>0-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4 - 7.9</td>
<td>&lt; 11</td>
<td>0-299</td>
<td>300-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 11</td>
<td>0-399</td>
<td>400-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 5</td>
<td>0-49</td>
<td>50-199</td>
<td>200-249</td>
<td>250-349</td>
<td>350-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 8</td>
<td>5 - 6.9</td>
<td>0-49</td>
<td>50-249</td>
<td>250-349</td>
<td>350-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 7</td>
<td>0-249</td>
<td>250-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>30-31.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 2</td>
<td>all</td>
<td>0-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2 - 3.9</td>
<td>&lt; 9</td>
<td>0-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4 - 7.9</td>
<td>all</td>
<td>0-299</td>
<td>300-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 8</td>
<td>&lt; 5</td>
<td>0-199</td>
<td>200-299</td>
<td>300-399</td>
<td>400-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>5 - 6.9</td>
<td>0-299</td>
<td>300-349</td>
<td>350-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 7</td>
<td>0-299</td>
<td>300-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>32-35.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 2</td>
<td>all</td>
<td>0-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2 - 3.9</td>
<td>all</td>
<td>0-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4 - 7.9</td>
<td>all</td>
<td>0-349</td>
<td>350-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 8</td>
<td>&lt; 5</td>
<td>0-199</td>
<td>200-299</td>
<td>300-399</td>
<td>400-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>5 - 6.9</td>
<td>0-299</td>
<td>300-349</td>
<td>350-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 7</td>
<td>0-299</td>
<td>300-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2.36</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 2</td>
<td>all</td>
<td>0-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2 - 3.9</td>
<td>&lt; 5</td>
<td>0-349</td>
<td>350-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>5 - 6.9</td>
<td>0-399</td>
<td>400-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>7 - 8.9</td>
<td>0-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 9</td>
<td>0-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 5</td>
<td>0-299</td>
<td>300-399</td>
<td>400-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 7</td>
<td>0-349</td>
<td>350-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4:1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 2</td>
<td>all</td>
<td>0-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2 - 3.9</td>
<td>&lt; 5</td>
<td>0-349</td>
<td>350-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>5 - 6.9</td>
<td>0-399</td>
<td>400-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>7 - 8.9</td>
<td>0-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 9</td>
<td>0-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 5</td>
<td>0-299</td>
<td>300-399</td>
<td>400-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 7</td>
<td>0-349</td>
<td>350-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>30-31.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 2</td>
<td>all</td>
<td>0-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2 - 3.9</td>
<td>&lt; 5</td>
<td>0-349</td>
<td>350-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>5 - 6.9</td>
<td>0-399</td>
<td>400-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>7 - 8.9</td>
<td>0-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 9</td>
<td>0-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 5</td>
<td>0-299</td>
<td>300-399</td>
<td>400-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 7</td>
<td>0-349</td>
<td>350-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>32-35.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 2</td>
<td>all</td>
<td>0-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2 - 3.9</td>
<td>&lt; 5</td>
<td>0-399</td>
<td>400-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>5 - 6.9</td>
<td>0-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 7</td>
<td>0-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 3 Culvert Treatment Recommendations for Low-Volume, Rural Roadways, B/C=2 (continued)

<table>
<thead>
<tr>
<th>Slope Rate SR</th>
<th>Road Width W (ft)</th>
<th>Drop Height H (ft)</th>
<th>Culvert Length L (ft)</th>
<th>ADT</th>
<th>Do Nothing</th>
<th>Remove Posts</th>
<th>Install Guardrail</th>
<th>Culvert Grate if wingwalls otherwise Remove Posts</th>
<th>Culvert Grate if wingwalls otherwise Install Guardrail</th>
<th>Culvert Grate</th>
</tr>
</thead>
<tbody>
<tr>
<td>4:1</td>
<td>32-35.9</td>
<td>4 - 7.9</td>
<td>&lt; 5</td>
<td>0-349</td>
<td>350-449</td>
<td>450-500</td>
<td>350-500</td>
<td>350-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 8</td>
<td>≥ 5</td>
<td>0-399</td>
<td>400-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>2 - 3.9</td>
<td>&lt; 5</td>
<td>0-449</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 36</td>
<td>≥ 5</td>
<td>0-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 4</td>
<td>all</td>
<td>&lt; 5</td>
<td>0-449</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4 - 7.9</td>
<td>5 - 6.9</td>
<td>0-449</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 8</td>
<td>≥ 5</td>
<td>0-399</td>
<td>400-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 30</td>
<td>6:1</td>
<td>30-31.9</td>
<td>4 - 7.9</td>
<td>0-449</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 8</td>
<td>5 - 6.9</td>
<td>0-399</td>
<td>400-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 4</td>
<td>all</td>
<td>&lt; 7</td>
<td>0-449</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4 - 7.9</td>
<td>&lt; 5</td>
<td>0-449</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 8</td>
<td>≥ 5</td>
<td>0-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 4</td>
<td>all</td>
<td>&lt; 7</td>
<td>0-449</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4 - 7.9</td>
<td>&lt; 5</td>
<td>0-449</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 8</td>
<td>≥ 5</td>
<td>0-399</td>
<td>400-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 4</td>
<td>all</td>
<td>&lt; 7</td>
<td>0-449</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4 - 7.9</td>
<td>&lt; 5</td>
<td>0-449</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 8</td>
<td>≥ 5</td>
<td>0-399</td>
<td>400-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 4</td>
<td>all</td>
<td>&lt; 7</td>
<td>0-449</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4 - 7.9</td>
<td>&lt; 5</td>
<td>0-449</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 8</td>
<td>≥ 5</td>
<td>0-399</td>
<td>400-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 4</td>
<td>all</td>
<td>&lt; 7</td>
<td>0-449</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4 - 7.9</td>
<td>&lt; 5</td>
<td>0-449</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 8</td>
<td>≥ 5</td>
<td>0-399</td>
<td>400-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 4</td>
<td>all</td>
<td>&lt; 7</td>
<td>0-449</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4 - 7.9</td>
<td>&lt; 5</td>
<td>0-449</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 8</td>
<td>≥ 5</td>
<td>0-399</td>
<td>400-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 8</td>
<td>all</td>
<td>&lt; 7</td>
<td>0-449</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>flat</td>
<td>&lt; 8</td>
<td>0-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 8</td>
<td>all</td>
<td>&lt; 7</td>
<td>0-449</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 8</td>
<td>all</td>
<td>&lt; 7</td>
<td>0-449</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 4  Culvert Treatment Recommendations for Low-Volume, Rural Roadways, B/C=4

<table>
<thead>
<tr>
<th>Slope Rate SR</th>
<th>Road Width W (ft)</th>
<th>Drop Height H (ft)</th>
<th>Culvert Length L (ft)</th>
<th>ADT</th>
<th>Do Nothing</th>
<th>Remove Posts</th>
<th>Install Guardrail</th>
<th>Culvert Grate if wingwalls otherwise Remove Posts</th>
<th>Culvert Grate if wingwalls otherwise Install Guardrail</th>
<th>Culvert Grate</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 30</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0-149</td>
<td>150-299</td>
<td>300-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.5:1</td>
<td>1.5:1</td>
<td></td>
<td></td>
<td>0-149</td>
<td>150-249</td>
<td>250-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0-199</td>
<td>200-299</td>
<td>300-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>30-31.9</td>
<td>4 - 7.9</td>
<td></td>
<td></td>
<td>0-149</td>
<td>150-249</td>
<td>250-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 8</td>
<td></td>
<td></td>
<td>0-199</td>
<td>200-249</td>
<td>250-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>32-35.9</td>
<td>&lt; 2</td>
<td></td>
<td></td>
<td>0-149</td>
<td>150-349</td>
<td>350-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 4</td>
<td></td>
<td></td>
<td>0-199</td>
<td>200-349</td>
<td>350-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 36</td>
<td>≥ 8</td>
<td></td>
<td></td>
<td>0-249</td>
<td>250-399</td>
<td>400-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2:1</td>
<td>&lt; 30</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4 - 7.9</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 8</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-399</td>
<td>400-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>30-31.9</td>
<td>&lt; 4</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4 - 7.9</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 8</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-399</td>
<td>400-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>32-35.9</td>
<td>&gt; 2</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4 - 7.9</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 8</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 36</td>
<td>≥ 8</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3:1</td>
<td>&lt; 30</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4 - 7.9</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 5</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-349</td>
<td>350-449</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>30-31.9</td>
<td>≥ 8</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-399</td>
<td>400-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 5</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-399</td>
<td>400-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>32-35.9</td>
<td>≥ 8</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 5</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 36</td>
<td>≥ 8</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4:1</td>
<td>&lt; 30</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4 - 7.9</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 5</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-349</td>
<td>350-449</td>
<td>450-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>30-31.9</td>
<td>≥ 8</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-399</td>
<td>400-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 5</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-399</td>
<td>400-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>32-35.9</td>
<td>≥ 8</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 5</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-449</td>
<td>450-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 36</td>
<td>≥ 8</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 6:1</td>
<td>&lt; 36</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 36</td>
<td></td>
<td></td>
<td>0-99</td>
<td>100-500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 5 Bridge Treatment Recommendations for Low-Volume, Rural Roadways, B/C=2

<table>
<thead>
<tr>
<th>Existing Rail Type</th>
<th>Offset (ft)</th>
<th>Length (ft)</th>
<th>Drop Height (ft)</th>
<th>Install Approved Bridge Rail (ADT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle Iron or W-beam</td>
<td>0 - 1.5</td>
<td>0 - 37.5</td>
<td>0 - 10</td>
<td>450-500</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>10.1 - 16.5</td>
<td>400-500</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>&gt; 16.5</td>
<td>350-500</td>
</tr>
<tr>
<td></td>
<td>37.6 - 75</td>
<td>10.1 - 16.5</td>
<td></td>
<td>450-500</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>&gt; 16.5</td>
<td>400-500</td>
</tr>
<tr>
<td></td>
<td>75.1 - 125</td>
<td>&gt; 16.5</td>
<td></td>
<td>450-500</td>
</tr>
</tbody>
</table>

Table 6 Bridge Treatment Recommendations for Low-Volume, Rural Roadways, B/C=4

<table>
<thead>
<tr>
<th>Existing Rail Type</th>
<th>Drop Height (ft)</th>
<th>Length (ft)</th>
<th>Offset (ft)</th>
<th>Do Nothing (ADT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle Iron or W-beam</td>
<td>all</td>
<td>all</td>
<td>all</td>
<td>0-500</td>
</tr>
</tbody>
</table>

ACKNOWLEDGMENTS

The authors wish to acknowledge several sources that made a contribution to this project: (1) the Midwest States Regional Pooled Fund Program for sponsoring this research project and (2) Marshall County officials for identifying low-volume roads in Kansas.

REFERENCES


<table>
<thead>
<tr>
<th>PAPER TITLE</th>
<th>Engineering Benefits of Pavement Management System Applications to Korea National Highways</th>
</tr>
</thead>
<tbody>
<tr>
<td>TRACK</td>
<td></td>
</tr>
<tr>
<td>AUTHOR</td>
<td></td>
</tr>
<tr>
<td>Jongeun BAEK</td>
<td>Senior Researcher</td>
</tr>
<tr>
<td>CO-AUTHOR(S)</td>
<td></td>
</tr>
<tr>
<td>Jae-Kyu LIM</td>
<td>Research Specialist</td>
</tr>
<tr>
<td>Tae-Hoon Lee</td>
<td>Research Specialist</td>
</tr>
<tr>
<td>Boo-Ill KIM*</td>
<td>Research Fellow</td>
</tr>
<tr>
<td>*E-MAIL (for correspondence)</td>
<td><a href="mailto:bikim@kict.re.kr">bikim@kict.re.kr</a></td>
</tr>
</tbody>
</table>

**KEYWORDS:**
Pavement management system, pavement performance, economic benefit, decision tree,

**ABSTRACT:**
This paper introduced a pavement management system (PMS) for national highways in Korea. The engineering benefits of the Korean national highway PMS applied for the last 20 years was evaluated in terms of cost and performance. The use of appropriate MR&R strategies to deteriorated pavements reduced the MR&R cost per unit length by approximately 75% and also enhanced the structural capacity of the pavements: pavement deflection was reduced by around 40%. Overall pavement performance was maintained properly: national highway pavement condition index (NPHCI) was reduced only 4.0%. This evaluation showed clearly that the application of the PMS in the Korean national highways was successful in terms of cost and performance.
Engineering Benefits of Pavement Management System Applications to Korea National Highways

Jongeun Baek, Jae-Kyu Lim, Tae-Hoon Lee, and Boo-II Kim*

Korea Institute of Civil Engineering and Building Technology, Geonggi-Do, Republic of Korea

*Email for correspondence: bikim@kict.re.kr

1. INTRODUCTION

Korean government has continuously invested to build road network in the last 30 years. As of 2013, Korea has 106,414 km of road network which consists of express highways of 4,111 km, national highways of 13,843 km, major cities of 19,555 km, and rural roads of 68,504 km (Statistics Korea, 2014). Most of the roads were paved: 100% of the express highways, 97.7% of the national highways, 99.3% of the major cities, and 73.5% of the rural roads. Recently, the road investment was shorten due to limited social overhead capital (SOC) budgets for road construction so that the main focus of road network was changed from building new roads to maintaining existing roads.

A pavement management system (PMS) is a tool to identify maintenance, repair, and rehabilitation (MR&R) strategies for damaged pavements and to determine project priority for efficient budget allocations (Saadatmand 2008). A PMS for efficient management of national highways was introduced in 1986 by Ministry of Land, Infrastructure and Transport (MOLIT) with an aid of Laboratoire Central des Ponts et Chaussées (LCPC). After that, the national highway PMS has been jointly operated by the government and Korea Institute of Construction Technology (KICT). Since 1999, KICT has conducted the national highway PMS alone. As of 2013, a total of 11,385 km of the national highways are managed by the national highway PMS. Also, Other PMSs have been also conducted to manage pavements in express highways, major airports, and large cities in Korea.

For the national highways PMS, various survey vehicles have been used to evaluate pavement conditions. At the beginning, manual-type of equipment such as deciroute, deflectograph, and skid resistance tester (SRT) was used for the national highway PMS. In 1997, state-of-art road survey equipment such as automatic road analyzer (ARAN) and falling weight deflectometer (FWD) were adapted. Currently, ARAN and PES (Pavement Evaluation Surveyor) are used to collect pavement distresses including cracks, patches, potholes, and rutting and pavement roughness. In addition, heavy weight deflectometer (HWD), ground penetrating radar (GPR), and pavement friction tester (PFT) were used to evaluate the structural capacity, layer thickness, and surface friction, respectively (Lim et al. 2013).

As listed, lots of PMSs have been used for various road levels in Korea, but the usefulness of the PMS has not been reported clearly. In this study, the engineering benefits of the national highway PMS applied for the last twenty years is evaluated in terms of pavement performance and cost efficiency.

2. PAVEMENT MANAGEMENT SYSTEM FOR KOREAN NATIONAL HIGHWAYS

Basically the national highway PMS follows a project-level approach to provide adequate maintenance, rehabilitation, and reconstruction (MR&R) strategies for selected sections as shown in figure 1. For efficient surveys, general pavement survey using ARAN and PES is conducted on a part of the national highways, e.g., 3,467 km (10,262 lane-km) in 2013, which was selected based on pavement conditions and maintenance history. For example, pavement sections which are seven years old or more, or have more than 20% of crack rate and/or more than 15 mm of rut depth should be included in the general survey. However, the following pavement sections are excluded when they are under construction, with good conditions, and were surveyed in details in the last year. Then visual index (VI) is calculated to represent pavement conditions in brief based on general pavement survey results of crack rate and rut depth. For all the sections excluding the first VI grade sections, detailed pavement survey is conducted on the detailed survey sections to evaluate structural capacity using FWD and to measure pavement thickness using GPR. Besides, PFT is used to measure pavement friction for the sections which were reported to have slippage problems. Then a proper MR&R method is determined based on the pavement conditions evaluation. First of all, MR&R budgets are allocated for prioritized repair sections whose pavement conditions are severe enough to be
repaired by the next year. For the other (general repair) sections, repair priority is decided with a net present value (NPV) computed by highway development and management model version 4 (HDM-4) which is a computer-aid decision making tool to check engineering and economic viability of the investments in roads (Kerali et al. 1998). After on-site due-diligence to do double-check for field conditions, the final MR&R method is determined.

Figure 1. PMS procedure and survey vehicles used for Korean national highways.

A decision tree used for the MR&R method for asphalt pavements is illustrated in figure 2. The first step in the decision tree is to select the prioritized repair sections where rut depth is greater than 15 mm and/or crack rate is greater than 30%. In the prioritized repair sections, four types of asphalt concrete (AC) overlay is applied depending on the pavement and traffic conditions of the sections. Cutting and applying 5-cm-thick polymer-modified asphalt (PMA) concrete overlay, in short Cut+5AC(PMA), is applied when 1) rut depth is greater than 25 mm or 2) rut depth is greater than 15 mm but not greater than 25 mm; and daily traffic volume is equal to or over 3,000 equivalent single axle loads (ESALs). Cutting and applying 5-cm-thick hot-mix asphalt (HMA) overlay, in short Cut+5AC(HMA), is applied when rut depth is between 15 mm and 25 mm and daily traffic volume is less than 3,000 ESALs. When rut depth is not greater than 15 mm but crack rate is greater than 30%, 5-cm-thick PMA concrete overlay, in short 5AC(PMA), is applied. Under the same condition, grid reinforcement can be applied at the bottom of 5-cm-thick HMA overlay, in short Grid+5AC(HMA), to mitigate reflective cracking when fatigue cracking exists more than 40% of surveyed area.

It is desirable to repair all the damaged sections, but only a part of sections can be repaired due to the limited budgets. So, in the second step of the decision tree, economic analysis is conducted to determine
the order of repair priority for the sections where rut depth is less than 15 mm and crack rate is greater than 10%. For these sections, 5-cm-thick HMA overlay with/out cutting is considered. In the economic analysis, both agent and user costs are considered during analysis period of 30 years. Recently, Korean version of HDM-4 was developed to consider various road, pavement, traffic, economy, and other conditions in Korea (Do et al. 2013). In this new program, rehabilitation timing and method can be altered for the next three years depending on pavement deteriorations. For example, 5AC(HMA) can be applied to a section in the next year, but if the repair is delayed for three years, more expansive method such as 5AC(PMA) or Cut+5AC(HMA) should be used because pavement conditions of the section would be worse. Thus, the new economic analysis can give use better solutions in terms of optimal repair method and timing. Finally, preventative methods such as slurry seal and microsurfacing can be applied to the sections where crack rate is less than 10%. In addition, recycling methods such as hot-in-place (HIR) recycling is recommended if applicable.

Figure 2. Decision tree to determine for MR&R methods for Korea national highways.

Major MR&R methods used for the Korean national highways in 2012 and 2013 are shown in figure 3. Among the methods, AC overlays made of HMA or PMA were popularly used: 62.2% in 2012 (48.3% for HMA overlay and 13.9% for PMA overlay) and 76.1% in 2013 (63.5% for HMA overlay and 12.6% for PMA overlay). Surface treatments and HIR recycling methods were used in 8.0% and 6.3% in average. The other methods include grid-reinforced overlay, concrete repair, and so on. In 2012, considerable budgets were urgently investigated to repair concrete pavements deteriorated by severe freeze-and-thaw phenomenon.
3. ENGINEERING BENEFIT OF PMS

3.1 Economic Benefit

The goal of the PMS is to maintain roads in a good condition within limited budget allocations. In order to ensure how much the national highway PMS works for that, total length of the national highways and annual MR&R costs for the national highways from 1991 to 2011 are compared in figure 3. The 2-lane length of the national highways increased by 91.2%; 13,081 km in 1991 and 21,345 km in 2011. On the other hand, annual MR&R cost of $62.0 million in 1991 was almost same as $64.6 million at 2011. So, if discount rate and/or cost per length is considered, annual MR&R cost was reduced significantly.
All the costs were converted to a value equivalent to 2011 based on annual inflation rate of 5%. In 1991 when PMS was introduced for the national highways, $125,700 per lane km was spent for MR&R; in 2011, only US $30,300 per lane km was spent, i.e., 75.9% of MR&R cost was reduced. It means that an appropriate MR&R methods were applied to right sections timely.

![Figure 5. Variations of annual unit maintenance cost at discount rate of 0.0% and 5.0%.](image)

3.2 Performance Benefit

3.2.1 Structural Capacity

From the national highway PMS database for the last 20 years, the maximum FWD deflection at the loading center, $D_0$ was obtained. As shown in figure 6, average $D_0$ of 0.43 mm measured before 2000 decreased to 0.25 mm (41.9% reduction) after 2000. It means that surface modulus of the pavement was enhanced by a factor of 1.7. This enhancement resulted from pavement thickness increase and lane extension. The main rehabilitation method applied for the last twenty years on national highways was AC overlay with/without milling. Thus, the progressive AC overlays thickened the AC layers from 10 cm to 20 cm or greater, i.e., this enhanced the structural capacity of the pavements. Also, national highways with two lanes were extended to four or more lanes, leading to a reduction of traffic volume per lane and reducing the rate of pavement damage accumulation.
3.2.2 Pavement Performance

The performance of the national highways is evaluated in terms of crack, rutting, roughness, and their combination. Since 2007, the pavement conditions of the national highways have been monitored to evaluate overall pavement quality. A total of approximately 2,300-km-long pavements were selected as for the performance monitoring sections, which is 20% of total national highways and a 1-km-long pavement section was surveyed every 5.0 km.

Figure 7 shows the variations of the pavement distresses in the national highways from 2007 to 2012. Rut depth kept decreased while crack rate gradually increased: rut depth was reduced by 27.1% from 2007 to 2012 and crack rate increased by 28.0% from 2007 to 2011. One of the reasons is that stiffer asphalt binder was preferred to use in the national highways since rutting seemed to be more critical at the middle of 2000. The other reason is that asphalt binder contents were reduced as much as mixture design can allows due to oil price increase. So, while the rutting problem could be minimized, cracking arised to be another problem and more serious was the significant occurrence of potholes. In addition, the increment of IRI from 2007 to 2012 was 32.6%. Since IRI was not considered for the MR&R method decision, IRI has not been recognized as an important parameter.
To evaluate overall pavement performance, NHPCI (national highway pavement condition index) was compared. Herein, NHPCI is a single parameter developed to represent overall pavement conditions by combining crack rate, rut depth, and IRI (Son et al. 2013). During the monitoring period, NHPCI decreased very slowly: Its maximum was 6.09 in 2008 and its minimum was 5.59 in 2012, only 4.9% reduction. So, overall pavement performance in the last 5 years could be similar.

Considering the cost effectiveness of the national highway PMS and the performance of the national highways, we can conclude that the national highway PMS helped obviously managing the national highways efficiently: increasing structural capacity, maintaining pavement performance as well as reducing MR&R cost.

4. CONCLUSIONS

In this paper, the engineering benefits of the application of the PMS on national highways in Korea for the last 20 years was evaluated in terms of cost and performance. The use of appropriate MR&R strategies to deteriorated pavements enhanced the structural capacity of the pavements by around 40%; maintained pavement performance in terms of NHPCI; and reduced the MR&R cost per unit length by approximately 75%. It is a clear evidence of the engineering benefit of the PMS.

5. ACKNOWLEDGMENTS

The authors appreciate the great supports from Ministry of Land, Infrastructure and Transport Pavement Management System.

REFERENCES


Traffic Flow Analysis by the Use of Wi-Fi Packets Receiver

Junji NISHIDA\(^1\) · Tomoyuki ADACHI\(^2\) · Kazuhiko MAKIMURA\(^3\)

\(^1\) Japan Research Institute for Social Systems (550-0002 1-10-27 Edobori Nishi-ku Osaka City, Osaka, Japan)  
E-mail:nishida@jriss.jp
\(^2\) Overseas Business Dept., West Nippon Expressway Company Ltd. (530-0003 1-6-20 Dojima, Kita-ku, Osaka City, Osaka, Japan)  
E-mail:t.adachi.ai@w-nexco.co.jp
\(^3\) Institute of Behavioral Sciences (162-0845 2-9 Ichigayaabonmura-cho, Shinjuku-ku, Tokyo, Japan)  
E-mail:kmakimura@ibs.or.jp

There has been sharp increase of mobile communication terminals such as smartphones and tablet computers with Wi-Fi function in use. When a smartphone performs packet communication, it transmits a unique ID number of the device at regular intervals even in standby mode, so it is possible to track the movement of mobile bodies by receiving packets at multiple locations on roadsides and analyzing them.

This paper shows the results of experiments analyzing traffic and person flows by the use of Wi-Fi packet receiver which was uniquely developed by the authors. The authors measure traffic flows on an expressway and ordinary road and calculate travel time both in Japan and abroad. Person flows at a service area on expressway and a fast-food restaurant in an urban area are also measured in order to analyze residence time and usage situation of the facilities.

The possibility of further applications to the breakdown of traffic-flow and/or low-cost Origin-Destination surveys by additional allocation of sensors in wider areas will be also discussed in the paper. The traffic-flow analysis using Wi-Fi packet data from smartphones should be more beneficial especially in developing countries with insufficient traffic-flow observation systems.

**Key Words**: system analysis, travel behavior survey, road planning, public transport planning, traffic information, ITS

1. Research Objectives and background

Due to the rapid dissemination of smartphones in recent years, many mobile bodies have become equipped with communications terminals that have Wi-Fi functionality. Since many smartphones transmit a type of Wi-Fi packet that includes a unique ID number for the device, by receiving and analyzing these packets, it is possible to ascertain the traffic flow of a broad range of mobile bodies.

By receiving Wi-Fi packets on roadside and comparing data at multiple locations, it is possible to measure the movement speed over distances (travel speed) of automobiles and people at extremely low cost. Furthermore, if we measure the start and end time of Wi-Fi packets that are continuously observed at the same location, we can measure residence time of people and automobiles.

These traffic flow sensors that can easily observe traffic flows from the side of the road could be utilized in many regions. In particular, they hold much promise for practical use in developing countries, where the facilities for traffic flow measurement infrastructure are insufficient.

In the past few years, a variety of approaches and research for implementing analysis methods of traffic flow through the acquisition of unique wireless packet information have been carried out in many places. However, many of these use Bluetooth, and not much research on Wi-Fi packets has accumulated. Moreover, in order to measure traffic flow by acquiring unique wireless packet information, we must use a method that pays the utmost care to protection of personal privacy, or else even if all of the technical problems were resolved, it would be difficult to gain the understanding of the public.

We developed the hardware and software for using Wi-Fi packet sensors to analyze traffic flow, applied them in multiple fields, and took many measurements. In addition to capable of measuring traffic flow of vehicles on expressways and ordinary roads, we were able to confirm that the Wi-Fi packet sensors could be effective as sensors to ascertain the flow of people in congested areas such as service areas and metropolitan areas. Moreover, making the acquired data anonymous and conforming to personal information protection laws is indispensable. We consider measures to delete the data from users who do not wish to be acquired (in other words, an “opt-out” method), and propose a method of operating the sensors in a way that does not cause any legal or socio-ethical problems.

2. Wi-Fi Packet Receiver

In recent years, mobile information devices with Wi-Fi communication functionality have rapidly disseminated. The terminal with the highest rate of dissemination is the smartphone, but other devices, from notebook PCs to mobile gaming devices and recently even digital cameras, are also outfitted with Wi-Fi communication
functionality.
Most of these devices transmit management packets, called “Probe Request Packets,” that search for Wi-Fi routers to connect to even in standby mode. There is some discrepancy between the frequency at which different devices transmit this signal, but most generally transmit them at intervals of around 30 to 90 seconds. Since this packet includes a device-specific address assigned to each device (a MAC address), by comparing the MAC addresses of packets acquired by sensors in multiple locations, we can analyze many types of traffic flows.

It is not possible to identify an individual solely based on the unique device information included in the packet. If, however, one were to obtain the MAC address of an individual and maliciously connect it to the person’s personal information, it would be possible to track that individual’s activity. So we convert the acquired MAC address inside the sensor through a one-way hash function so that the analysis can be processed anonymously.

We call this sensor, which receives Probe Requests with anonymous MAC addresses, the Anonymous MAC address Probe sensor, or AMP.

3. Literature Review

(1) Traffic Flow Analysis using Bluetooth packets
It is possible to measure travel speed of automobiles by receiving packets transmitted from Bluetooth devices in automobiles and comparing them with MAC addresses as identifiers. Much traffic flow analysis research using this method has been presented since 2010, particularly abroad. Trung Vo used this method to measure travel speed of automobiles on trunk roads. Compared to measurement methods using license plate identification, magnetic sensors or RFID tags, the measurements were effective with respect to accuracy, cost superiority, and anonymity. He has conducted various other researches on analysis of traffic flow with this method.

In Japan, Kitazawa and Shiomi have attempted research to measure travel speed on the Hanshin Expressway, and they succeeded at measuring travel speed by comparing the MAC addresses of packets transmitted via Bluetooth measured at two points on the Hanshin Expressway.

(2) Traffic Flow Analysis using Wi-Fi packets
Compared to analysis of traffic flow using Bluetooth, there is little research on analysis of traffic flow using Wi-Fi packets.

Luber Andreas et al. compared measurements of travel speed on roads using Bluetooth and Wi-Fi and conducted research to reevaluate the superiority of each. However, upon setting up Wi-Fi and Bluetooth wireless devices on roadside gantry and comparing the detection of each, they found 6.5% detection of Bluetooth and only 1% detection of Wi-Fi, leading to the conclusion that Bluetooth was favorable. However, in fields other than roads, there is research by Ryusuke Nakano and others on approximating the degree of crowding inside train cars by attempting to estimate the number of passengers inside the cars using probe request receivers installed inside the cars.

(3) Comparison of Bluetooth and Wi-Fi System
As stated above, based on the existing research, there are many examples concluding that measurements by Bluetooth are more favorable than those by Wi-Fi.

The reason for this is that with Bluetooth measurements, first the sensor transmits an API connection request (a request for coupling), waits for the other terminal to respond, and then acquires the MAC address included in the reply from the other device, so acquisition of the other device’s MAC address is simple. Furthermore, we can infer that since the dissemination of Bluetooth preceded Wi-Fi for transmission for devices inside automobiles, such as mobile telephone headsets and devices to connect to the sound system, the result of the measurement was that the rate of detection of Wi-Fi was higher than that of Wi-Fi.

However, in analyzing traffic flow on expressways, when measuring with Bluetooth, it takes a certain amount of time (8 – 13 seconds) between transmitting the connection request and reply from the device. So during that time the automobile may leave the sensor’s effective range and it has been pointed out in detection of high-speed vehicles, the detection rate drops. In the aforementioned research by Kitazawa et al., the number of MAC addresses detected at both points on the Hanshin Expressway in Japan was quite small, at only 2 – 17 in a 2 hour interval. A cause of this was, in addition to the response time for coupling, the problem of the wave strength approved for Bluetooth. Namely, in other countries the wave strength approved for Bluetooth is 100mW, but in the Japanese Radio Law only half that strength, 50mW, is approved.

On the other hand, the recent rapid dissemination of smartphones has drastically increased the number of automobiles with Wi-Fi devices in a short time. In an article in ITS World presented at approximately the same time as our research introducing the measurement results of Blip Systems A/S from the Netherlands, it was reported that although previously the detection rate for Bluetooth was clearly higher, recently the detection rate for Wi-Fi has exceeded that of Bluetooth.

It is not a matter of whether to use detection by Bluetooth and detection by Wi-Fi, but by using a hybrid sensor that responds to both types of signals, it is possible to analyze traffic flow with a higher detection rate. Blip Systems A/S of the Netherlands already developed a sensor that measures both types simultaneously.
4. Organization of the System

To develop the AMP sensor, we aimed to make it easily installed in a variety of environments and developed the following two items.
A. Development of sensor software that could be operated on general-purpose PCs
B. Development of low-cost, small sensor hardware

(1) Development of Sensor Software Operated with General PC

We have completed development of sensor software that operates on three different operating systems (OS), Macintosh, Windows, and Linux.

The structure of the AMP sensor software is described in Fig. 1. After capturing the Wi-Fi packet, it is immediately hashed with SHA-1 and after storing the acquired log on the internal memory, it is uploaded to cloud storage server each designated period of time.

The time for each sensor is synchronized with NTP, so since the timestamps of multiple AMP sensors are synchronized, it is possible to accurately measure speed with the data from AMP sensors between multiple points.

(2) Development of Affordable and Small Sensor Hardware

We used hardware, Raspberry Pi Model B. The CPU is an ARM 1176JZF-S 700MHz. The storage is SDRAM 512MB. The cabled connection is a 10/100 BaseT Ethernet socket in the main unit, which uses USB wireless LAN adapter connected via USB 2.0 for wireless signals. It consumes 3.5W of electricity, and the OS is the Linux-based Raspbian 3.6.1+

When the unit is connected to the Internet, such as via Ethernet cable, it automatically uploads the recorded data to a cloud storage server. However, when it is set up autonomously, it records its log on an SD card and can continue operating for about one month. Furthermore, in order to equip it with UPS function (Uninterruptible Power Supply), when operating on a mobile battery, it can function for approximately 3 hours even without electricity.

The unit was intended to be installed indoors. Thus, in order to set up outdoors, it needs to be produced and take into consideration of temperature changes and weather resistance such as using industrial circuit board.

(3) Cloud Storage Server

The West Nippon Expressway Company Ltd. (NEXCO-West) has cloud storage servers that can be accessed via the Internet for the purpose of collecting, analyzing, and utilizing a large amount of probe data. It is simple to scale up this cloud storage server by increasing the amount of data, and it has a distributed database processing engine that can process large quantities of data.

The collected data transmitted from the AMP sensor records the SHA-1 hashed MAC address, time received, and signal strength in a single line and stores the acquired log on the server each designated period of time. The amount of data transmitted from one AMP sensor is not large, but in the future, in order to smoothly process data if many AMP sensors are set up and continuously operated, we developed the environment capable to use NEXCO West’s high-powered cloud storage server.

The data uploaded to the server is immediately aggregated by the analysis system, and from the difference in time between the observations of the same ID measured at two designated locations, the distribution of travel speed for each designated period between each section is calculated.

(4) Wi-Fi Packets Accessible Distance

Using the system explained in (1) - (3), we conducted various functionality tests of the AMP sensor operation and data processing. The Wi-Fi probe packets had the quite significant travel distance of 200 – 350m. However we measured the signal strength of each packet, and it was possible to judge the distance using this signal strength (-35db – -95db). We have just begun to consider the possibility of structure that identifies location based on signal strength.
by installing multiple AMP sensors at designated distances, and it will be the subject of future research.

5. **Traffic Flow Analysis on Roads**

We measured travel speed on roads at following locations using either the AMP sensor we already developed or AMP sensor prototype under development.

A) Japan: Packet Acquisition test on National Route 163
   Date/Time: Multiple days in April, 2013

B) Japan: Travel speed measurement on the Sanyo Expressway
   Date/Time: July 5, 2013 11:50 - 12:50

C) Japan: Residence time measurement at Miki Service Area on the Sanyo Expressway
   Date/Time: July 5, 2013 13:35 - 14:25

D) Japan: Travel speed measurement in Osaka
   Date/Time: January 12, 2014 10:00 - 12:00

E) Indonesia: Travel speed measurement in Makassar
   Date/Time: January 22, 2014

In this article, we will discuss the measurement results of the first test conducted, “B) Japan: Travel speed measurement on the Sanyo Expressway,” as well as “E) Indonesia: Travel speed measurement in Makassar”.

1. **Measuring Travel Speed of Section on Sanyo Expressway**

At the beginning stage of development of the AMP sensor, using the sensor software developed for the internal Wi-Fi antenna in general-purpose PCs (Macintosh, MacBook Air, Linux PC), we installed the devices over an 11.4km interval on the Sanyo expressway between Miki-Higashi and Kakogawa and conducted test measurements to see if it would be possible to receive Wi-Fi packets from high-speed vehicles and to measure travel distances in that section by comparing MAC addresses.

Before this experiment, in the preliminary experiment conducted along the side of an ordinary road, we were able to confirm receipt of a certain number of unique IDs, but we conducted this experiment to confirm whether it would be possible to measure the travel speed of high-speed vehicles. The section measured in this experiment is shown in Fig. 3.

The measurement was conducted to monitor packets using MacBook Air, Linux notebook PC, etc. set next to a parked car on roadside (Fig. 4). For this reason, there were problems of bias in the packet acquisition according to driving direction. Since we did not set up a special antenna, the reception sensitivity was low as well.

![Fig. 4 Scenes of the Experiment on the Sanyo Expressway](image)

However, even though we employed this simple measurement method, as seen in **Table 1**, in 60 minutes of measurement we were able to acquire 134 samples on the Miki-Higashi side, and 139 on the Kakogawa side. We had 22 valid samples that were detected at both sites.

Table 2 shows the vehicle travel speed calculated using the acquired data. They almost match with the speed measured by the traffic counters installed in nearby locations shown in **Table 1**. Samples 20 – 22 had extremely slow speed of 19.1 – 45.6 km/h, but since there is a service area between these two points, it can be assumed that these vehicles stopped there. Suppose the average speed is 80 km/h, the amount of time spent at the Miki Service Area would be between 6 and 27 minutes.

Through this experiment, we were able to see that Wi-Fi packet sensing is well suited to the measurement of the travel speed of vehicles on expressways and ordinary roads.

**Table 1 Acquired Packets on the Sanyo Expressway**

<table>
<thead>
<tr>
<th>Location</th>
<th>Number of vehicle</th>
<th>Average speed in traveling lanes</th>
<th>Average speed in passing lanes</th>
<th>Acquired packets</th>
<th>Acquisition rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Miki-higashi</td>
<td>1,092</td>
<td>87.0km/h</td>
<td>103.9km/h</td>
<td>134</td>
<td>12.20%</td>
</tr>
<tr>
<td>Kakogawa</td>
<td>1,247</td>
<td>87.6km/h</td>
<td>105.4km/h</td>
<td>139</td>
<td>11.10%</td>
</tr>
<tr>
<td>Commonly acquired packets</td>
<td>22</td>
<td></td>
<td></td>
<td></td>
<td>1.76%</td>
</tr>
</tbody>
</table>

Note) Number of vehicle and travel speed by lane: traffic near the observation point
Value measured with counter (provided by NEXCO-West)
Table 2  Travel Speed Calculated from the Acquired Data

<table>
<thead>
<tr>
<th>Sample</th>
<th>Time difference (seconds)</th>
<th>Traveling speed (km/h)</th>
<th>Miki SA (minutes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>342.481</td>
<td>119.8</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>356.781</td>
<td>115</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>369.128</td>
<td>111.2</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>374.231</td>
<td>109.7</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>395.383</td>
<td>103.8</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>407.784</td>
<td>100.6</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>408.896</td>
<td>100.4</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>441.045</td>
<td>93.1</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>443.945</td>
<td>92.4</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>444.562</td>
<td>92.3</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>447.322</td>
<td>91.7</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>453.151</td>
<td>90.6</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>458.658</td>
<td>89.5</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>460.465</td>
<td>89.1</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>484.484</td>
<td>84.7</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>488.563</td>
<td>84</td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>494.112</td>
<td>83.1</td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>516.353</td>
<td>79.5</td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>525.361</td>
<td>78.1</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>550.668</td>
<td>65.6</td>
<td>6.3</td>
</tr>
<tr>
<td>21</td>
<td>1,011.09</td>
<td>40.6</td>
<td>8.2</td>
</tr>
<tr>
<td>22</td>
<td>2,153.62</td>
<td>19.1</td>
<td>27.2</td>
</tr>
</tbody>
</table>

There were frontage roads on the expressways measured, and there were both four-wheeled automobiles and two-wheeled motorcycles driving on the frontage road in the a1-a2 section. In particular, there were many section s of the frontage road in the a2-a3 section that were narrow and were only for two-wheeled vehicles. Two-wheeled vehicles (motorcycles) were not permitted on the trunk road of the expressway.

Since, as noted in 4(4), the traveling distance of the Wi-Fi packets was 200 – 350m, there was the problem that the AMP sensor set on the trunk road would measure the travel speed of vehicles driving on the frontage road as well.

Next we will explain the situation of section 2. Section 2 in Fig. 5 (measurement points b1, b2, and b3) is a trunk road from the airport to the city center. It is a trunk road with mostly one lane traveling in each direction, and there are commercial facilities alongside the road. Section 3 (measurement points b4, b5, and b6) is a trunk road with 2 or 3 lanes of traffic in either direction, but it is near the city center and there are many traffic jams mainly during peak hours in the morning and evening.

This means that on ordinary roads, measurements of travel speed
would not be only of automobiles, but of bicycles and pedestrians as well.

![Fig. 7 State of Traffic on Ordinary Road (b5-b6 Section)](image)

Table 3 shows the results of measurements taken by the AMP sensor in sections 1 – 3.

**Table 3 Measurement Results in Makassar**

<table>
<thead>
<tr>
<th>Section</th>
<th>Point</th>
<th>Number of vehicle during observation</th>
<th>Observed number of unique ID</th>
<th>Observation time (min)</th>
<th>Number of unique ID per minute</th>
<th>Acquisition rate (%)</th>
<th>Observed number of unique ID at 2 points</th>
<th>Number of observed unique ID at 2 points per minute</th>
</tr>
</thead>
<tbody>
<tr>
<td>High way</td>
<td>a1</td>
<td>2,025</td>
<td>143</td>
<td>5.6</td>
<td>7.1%</td>
<td>1.74</td>
<td>0.82</td>
<td></td>
</tr>
<tr>
<td></td>
<td>a2</td>
<td>3,177</td>
<td>318</td>
<td>3.5</td>
<td>10.6%</td>
<td>1.94</td>
<td>1.04</td>
<td></td>
</tr>
<tr>
<td></td>
<td>a3</td>
<td>3,258</td>
<td>173</td>
<td>3.8</td>
<td>5.3%</td>
<td>1.30</td>
<td>0.53</td>
<td></td>
</tr>
<tr>
<td>Ordinary East</td>
<td>b1</td>
<td>4,512</td>
<td>158</td>
<td>2.6</td>
<td>3.5%</td>
<td>1.56</td>
<td>0.93</td>
<td></td>
</tr>
<tr>
<td></td>
<td>b2</td>
<td>6,456</td>
<td>246</td>
<td>4.1</td>
<td>3.7%</td>
<td>1.83</td>
<td>1.36</td>
<td></td>
</tr>
<tr>
<td></td>
<td>b3</td>
<td>5,898</td>
<td>217</td>
<td>3.6</td>
<td>3.7%</td>
<td>1.43</td>
<td>1.04</td>
<td></td>
</tr>
<tr>
<td>Ordinary East</td>
<td>b4</td>
<td>12,738</td>
<td>471</td>
<td>7.9</td>
<td>3.7%</td>
<td>1.71</td>
<td>2.85</td>
<td></td>
</tr>
<tr>
<td></td>
<td>b5</td>
<td>12,120</td>
<td>761</td>
<td>12.7</td>
<td>6.3%</td>
<td>2.17</td>
<td>3.62</td>
<td></td>
</tr>
<tr>
<td></td>
<td>b6</td>
<td>13,794</td>
<td>459</td>
<td>7.7</td>
<td>3.3%</td>
<td>1.77</td>
<td>2.97</td>
<td></td>
</tr>
</tbody>
</table>

*Unique ID is the number of unique ID included in Wi-Fi packet measured at each observation location. It matches the number of observed Wi-Fi devices.

*Number of vehicles on ordinary roads includes the number of two-wheelers (motorcycle).

*“Highway” observation location is near the toll road exit, and vehicles tend to slow down.

*“b5” observation location is near a commercial complex.

Based on these results, we can see that the capture rate is low compared to the measurements in Japan in July, 2013 on the Sanyo Expressway. The number of IDs that were measured at two locations, and the number of observations that could be used to measure travel speed was average of 0.8 to 1.0 per minute on the expressway, and 0.9 to 3.6 per minute on ordinary roads. In a time period with similar levels of traffic, if setting the time units for travel speed measurement at 15 minutes, it is possible to acquire above average sample numbers in 15 minutes on both inbound and outbound routes on the expressway. Locally, the rate of dissemination of smartphones is increasing, so we expect that the accuracy of this measuring method of travel speed will improve in the future. Furthermore, as at location a2, near the tollbooth on expressways where drivers slow down, the capture rate improves. If we install sensors at these locations giving priority, the capture rate would increase further.

Next, we calculated the traveling time between two locations from the packets that could be measured at two points. At each measured section we looked for the distribution of travel speed. The measurement results for the a2-a3 section are shown in Fig. 8.

![Fig. 8 Travel Speed Measurement Results on Expressway (a2-a3 Section)](image)

The lighter bar shows the inbound route (from the airport into the city center), and the black bar shows the outbound route (from the city center to the airport). Overall, the reason that the number of observations of inbound vehicles was greater is thought to be because the observing vehicle was parked along the inbound side of the road to take measurements. We interpret the reason that the distribution is divided in two to be that the lower speed group was vehicles traveling on the frontage road, and the higher speed group was vehicles traveling on the expressway. As there was tropic squall about 10 minutes after beginning the measurements on the expressway, heavy vehicles that slip easily, such as container trucks lowered their speed. The reason that the speed distribution of vehicles diving on the expressway was wide, from 35 – 75 km/h, is due to the effects of the squall.

Next, the results of the b5-b6 section from among the travel speed measurements of the ordinary roads are shown in Fig. 9.
In Makassar, there are still few traffic lights on ordinary roads, and thus, the speed is high without traffic jams, 40 – 50 km/h. However, during rush hours, cars attempting to turn right stop the flow of oncoming traffic, and intense traffic results. The time when the measurements were taken overlapped with the evening rush hour, and especially on the outbound lane where people were returning home, a decrease in travel speed due to traffic was measured.

6. Residence Time Analysis

In the traffic flow analysis by the AMP sensor, in addition to analyzing travel speed between two points, by aggregating the time records when the same ID was measured continuously, it is possible to find distribution of residence time. On the expressway, it is effective in analyzing usage tendencies and residence time of service areas and parking areas.

We will next introduce an example of this residence time analysis.

(1) Distribution of residence time at the Miki Service Area

On the same date and time that we measured travel time on the Sanyo Expressway between Miki-Higashi and Kakogawa as explained in 5(1), we measured the residence time distribution at the food court in the Miki Service Area. The results of these measurements are shown in Fig. 10.

The food court at the Miki Service Area is located next to retail stores but is distant from restaurant and toilet. In these results, we see that during the 50 minutes that measurements were taken, we measured about 60 samples. Here, we defined delays as those samples measured continuously for over one minute. Samples that did not stay inside the building and just passed through in front of the buildings were excluded.

Since we started at 13:35, after lunchtime, but we observed many people eating at the service area. However, we also observed many who purchased drinks from vending machines, browsed quickly through the store, and left. The results in Fig. 10 show that many of the users during the time at the service area facility were there for only a short period of time.

(2) Distribution of residence time at fast food restaurant in front of the train station

This example of analysis is not related to roads, but with the cooperation of a fast food restaurant in front of the train station, we took continuous measurements over the course of 17 days. We are unable to publicize the name of the restaurant and the date and time measurements were taken, but are able to use the measurement data taken exclusively for academic purposes.

the number of customers at the point-of-service (POS) registers and the number of AMP sensor measurements for five of those 17 days including a weekend. Based on these results, we can see that the number of customers counted at the POS registers in the fast food restaurant in front of the station matches accurately the number of customers staying in the restaurant measured by the AMP sensor. The correlation coefficient between these two numbers was 0.945.

There were some discrepancies, depending on the hour, between the number of customers at the POS registers and the numbers measured by the AMP sensor; the numbers for customers at the POS registers is higher during lunchtime on weekdays, and the numbers observed with the AMP sensor were higher in the evenings. As this restaurant locates in suburb, there were many groups of middle-aged housewives during the day, and there were many students and businessmen in the evenings. Therefore, we infer that the difference in ownership rate of smartphones among customer types and the hours affected the results.
7. Sensing and Personal Information Protection

In the AMP sensor system, we took the utmost care to protect personal information; the measured MAC addresses were made anonymous using hashing function, and the coding and hacking countermeasures on the server were set at high level.

The information acquired with this system is not categorized as personal information stipulated under the Japanese Personal Information Protection Law, but it has characteristic of personal data that can be used to recognize individual's behavior. Currently in Japan, there will be amendment of the Personal Information Protection Law, and a bill will be submitted in 2015. In advance of the amendment, in December 2013, the “Policy Outline of the Institutional Revision for Utilization of Personal Data” related to personal data protection was publicized. Following this policy outline, we developed principle on the system taking the following measures:

1. Define the purpose of use for data collected
2. Define the content of data collected and how it is to be handled
3. Define prevention measures for those who do not wish to be observed (such as by turning off the Wi-Fi functionality on their smartphone)
4. Define the contact information for deleting data if anyone's personal data was obtained against their wishes, and preparation method for handling (opt-out measures)
5. Set a period of data storage for analysis, and clearly state to not disclosing data to any third parties

If measurements were to be taken in small area, the above 1. to 5. could be posted around the sensor, etc. If it is wide area, they could be posted on website or issued a press release. It would be necessary to publicize the information with appropriate methods.

8. Conclusion

In this article, we described the sensor software and hardware to conduct analysis of traffic flow by sensing Wi-Fi packets from smartphones, etc., that have been disseminated rapidly in recent years. We also showed applications in measuring travel speed and residence time analysis. The AMP sensor we developed shows high potential to be used for analysis of traffic flow in vehicles on roads including expressways, and people flow. Furthermore, we proposed measures to protect personal information and personal data, which are both legal and ethical concerns once the system is implemented in practice.

Wi-Fi packet sensors are a measurement method that has become practical rapidly in the past one to two years. We expect that it can be used widely in society.

In the future, we hope to continue with not only analyzing travel speed and residence time at a few locations as in this article, but also by installing numerous sensors widely, we hope to continue with systems to examine wide-area traffic flow and applications for Origin-Destination surveys at low cost and so on. Moreover, we hope to further research in various fields, such as disaster planning and operations management of urban facilities, etc.

Acknowledgement: Section 4. of this research was carried out under the commission of the Strategic Information and Communications R&D Promotion Programme (SCOPE) (Receipt number 132307011). Furthermore, the results of section 5. (2) are the results of the research project funded by the New Energy and Industrial Technology Development Organization (NEDO). We wish to express our appreciation for them.

Reference

1) Trung Vo : An Investigation of Bluetooth Technology for Measuring Travel Times on Arterial Roads: A Case Study on Spring Street ; A Thesis Presented to The Academic Faculty Georgia Institute of Technology, May 2011
4) Luben Andreas, Junghans Marek, Bauer Sascha, Jan Schulz : On Measuring Traffic with Bluetooth and Wi-Fi , 18th ITS World Congress, 2011
6) Stevanovic Aleksandar, Olarte Claudia L, Galletbeitia Alvaro, Galletbeitia Borja, Kaisar Evangelos I : Testing Accuracy and Reliability of MAC Readers to Measure
8) Strategic Headquarters for the Promotion of an Advanced Information and Telecommunications Network Society: Directions on Institutional Revision for Protection and Utilization of Personal Data, December 20, 2015
The Impact of Road Network Development on Land Use: Case Study Karawang Regency

KEYWORDS:
Land use, network development, Karawang Resident

ABSTRACT:
Road network planning could not be separated with land use planning or spatial planning. Literature from urban development research invented what is called by the land-use transport feedback cycle. The idea of the cycle is improvement of accessibility could attract investment of land therefore the land-use will change. The change of land-use than could increase activity and traffic and therefore reduce transport capacity. The paper tries to analyse the current data about the Gross Domestic Regional Product (GDRP) of Karawang Regency related to road network (Jakarta-Cikampek toll road). It is trying to find out the dominant factors that affect land use management relative to industrial activity and mobility patterns. It will discuss the importance of land use management in Karawang Regency along the corridors of the toll roads by considering the issues of fast-beyond-expectation growing areas and changes in land use in several locations along the road network. It concludes that in the land-use is change from agricultural to industrial area impacted by the provision of new road network. It also recommends that well-planned of land use in the toll road corridor could avoid the negative impacts in the future.
The Impact of Road Network Development on Land Use:  
Case Study Karawang Regency

Alfa Adib Ash Shiddiqi, ST. MSc  
Irnanda Satya Soerjatmodjo, ST. MSc  
Planning Division BBPJN IV, Directorate General of Highways

Email for correspondence: a.a.ash-shiddiqi@bbpjn4.com  
Submitted for the 1st IRF Asia Regional Congress & Exhibition, Bali November 17-19, 2014

1 INTRODUCTION

Regional development is influenced by the economic activities as showed by income per capita. It is believed that adequate infrastructure facilities are one of the important supports in economic. Transportation system is then can be considered as a “backbone” of the regional economic growth.

Intercity trips in Indonesia has been very massive as much as 3.8 billion trips per year, which are 74% of them are in Java Island; in addition cargo transport in the region also reach 1.8 million tons cargo or 75% of the total nationwide (Parikesit, 2004).

Arterial road network in the Java Island have strategic role, economic growth need to be supported by increase in road networks including toll roads. The development of toll roads has advantages such as: (i) Increasing accessibility which supports development of industrial areas and settlements, (ii) Increasing economic activities in local areas surrounding, (iii) Toll roads will play an important role in increasing road capacity immediately.

Latest data, Indonesia has already had 606-km toll roads, in which 76% of them are operated by PT. Jasa Marga and leaving the remaining proportions to private sectors. The toll road branches to be projected in development by 2009 are 1,593 kilometers, consisting of 293 km Trans-Java urban toll road (main trunk) and 56 km outer-Java toll road (Department of Public Works, 2004).

Toll road development is likely to change the urban environment, including changes in individual behavior and socio-economic structure. It also affects area planning and transportation facilities (Tamin and Russ, 1997). The development of highway transportation has created a major change (expected to be development) in the surrounding area.

Typical phenomenon found in cities or urban areas in Indonesia connecting national activity center, including the Jakarta - Cikampek route that passes Karawang Regency, is the new installment of toll roads. Karawang Regency is regency which the intersection between Jakarta Surabaya and Jakarta-Bandung is located in Cikampek (see Figure 1).

The presence of the toll roads changes urban spatial structure and affects urban area development at the left and right sides of the toll corridors, such as: (i) changes in spatial function, from agricultural area to industrial area, settlement, or other commercial sites, (ii) urbanization from rural area to the city, (iii) Emerging of new settlements in order to pursue the centers of employment. Many cities have changed themselves from small towns to industrial urban areas because they now become accommodated and facilitated by the presence of the toll roads. Such condition accelerates the movement of changes in social, economical, and political aspects.
Scope of the paper is discussing the importance of land use management in Karawang Regency along the corridors of the toll roads by considering the following issues: (i) Rapid growing the development of the area along the toll roads, which is beyond the expectation and causing unbalanced between the capacity of road and the traffic demand, and (ii) changes of land use at some location along the road network.

This paper is aiming to: (i) find out the influence of toll road development on urban land use and road network development pattern by considering characteristic of the area and local people's activity and mobility patterns based on their socio-economic conditions such as the Gross Domestic Regional Product (GDRP) of Karawang Regency, (ii) find out the influence of toll road development on industrial and economic development, (iii) identify correlation between transportation planning and spatial management plan.

The paper will be presented with introduction followed by some literature and form of method, and then the result of data analysis will be explained to support the conclusion and recommendation.

2 LITERATURES

This segment will briefly discuss some literatures in the field of transportation and urban development and planning.

Transportation

Transportation is a process of moving goods from one area to another in massive numbers, in safety, smooth, and timely manners. It can be performed if it is supported by adequate transportation facilities. The advance of transportation technology allows movement of goods (and people) from one area to another in the right place (place utility) and at the right time (time utility) so that their values become higher as the transportation cost is lowers (Morlok, 1995).

The role of transportation related to urban problems is very important. Road traffic in urban area can be controlled by transportation management, which supports society goals by collective and cooperative transportation activities. The government involves private sectors to make joined efforts to create freeway access for transporting products, goods, while still wary of the balance of settlement, industrial site, and other business activities (Oglesby and Hicks, 1982).

Research in regional science recognized that both growth and concentration of economic activity at any given location depends on access to markets and the location economies enabled by that access (Weisbrod, 2007). Economic life of a region may not operate without facilities, such as structures and infrastructures. Infrastructure of particular region or area can be an indicator for area development, adding in income per capita. There is a strong relationship between available infrastructures. For example, national economic
activity centre (Pusat Kegiatan Nasional/PKN) is where economic transactions, such as market, shopping centres, and restaurants take place, and where supporting infrastructures are situated.

Jakarta - Cikampek Toll Road, which is part of Java North Coast traffic networks that plays an important role in economic development and distribution, is used as a case study. The importance of the Jakarta - Cikampek Toll Road to the Java North Coast traffic networks is act as a national-class road with primary arterial function, it connects cities that are national activities centres (PKN), including Jakarta, Semarang, and Surabaya and acts as the connector between national leading sites.

Prior to the construction of Jakarta-Cikampek toll road, Karawang Regency was known as a rice producer. After the toll road has been operated, it becomes the site for factories, which lie at the right and left side of the road. There is a change in land use from agricultural area to industrial site. Local people also change their living from being farmers to factory labors. According to result of the economic monitoring in Karawang Regency in 1983-1991, agricultural contribution to the regency’s gross domestic income during the 1983-1991 tended to decrease. In 1983 the agricultural contribution was 38.83%, whereas in 1991 it only contributed as far as 29.78%.

Industrial sector holds more significant role to the economy of Karawang Regency by increasing the contribution to the gross domestic income from 11.54% (1983) to 18.16% (1991). In 2003, the regency held another economic monitoring that reported decreasing role of the agricultural sector for the gross domestic income. On the other hand, industrial sector became more important to the regency.

Urban Development

Urban spatial management structure is formed by market power that interacts with regulations and primary infrastructure investment (Alain Bertaud, 2004). Urban spatial management structure can be generally defined as an accidental and unexpected result of policies and regulations made without considering the spatial management (Soetomo, 2004).

Types of proper urban structure must allow labor mobility in the metropolitan area. Wherever the family live in this area, they must be reached in a reasonable length of time (i.e., less than one hour) from any working location to be offered.

![Figure 2 Connection Between Land Price and Distance to the City Centre (Source: Alonso 1964)](image)

William Alonso (in 1964) discussed a correlation between location and urban land use. His analysis based on economic rent and location rent concepts, following Von Thunen’s (1926) idea by formulating several assumptions. There are four assumptions in order to support the theory: a city has only one centre (CBD), any work sector is situated in the CBD, and each buy-sell activity only took place at the CBD.
All that occur in the city have the same advantage. Transportation costs are linearly related to distance. Transportation cost towards the city centre increases by the distance towards it. CBD is considered as the area with the highest reach, in which the further distance from the CBD, the lower the accessibility rate. Each land is sold to the highest bidder. It means that all parties have equal opportunity to get the land and monopoly is inexcusable in the “land market”, either from buyer’s or seller’s view.

According to the above assumptions, urban area market will take the form similar to agricultural area market as Von Thunen suggests. Each land location is sold to the highest bidder (see Figure 2).

- The Best Use of Land is a function that is capable to get the biggest return from the location.
- Bid-Rent Curves relates to pattern of urban area land use: retailing, industrial, and residential.
- Retailing has the steepest bid-rent curve because it considers accessibility where this function needs the highest accessibility rate. Bid-rent curve for industries is quite gently sloping than those of retailing. Even though accessibility also determines industrial activities, its role is less important than that of retailing because many industrial products tend to be sold outside the city.

Urbanization process as the process of change or modernization will create heterogeneity inside and outside the cities. The following scheme of urbanization diffusion shows a model of developmental process from the core to peripheral areas. Urbanization theory is a theory of core-periphery development and Friedman also develops a model of four-stage area development (Potter et al., 1998) (see Figure 3).

![Figure 3 Summary of Friedman's Core Periphery Model (Source: Potter 1998)](image)

Urbanization process as the development of area growth centres in the above scheme is theoretical. For third world countries it hardly occurs. The most critical stage is the shift from 2nd to 3rd stage due to extension of national cities that creates Urban Primacy and Mega Urban Phenomenon, which becomes an extending metropolitan.

### 3 METHODOLOGY

The study has been developed based on urban design and transportation knowledge. Various kinds of studies have been widely investigated in the previous time using the similar technique. The first step of this research is identifying the problems due to the relevant aspects in order to conduct the research method.

This research study includes 5 steps: (1) problem identification, (2) literature review, (3) secondary data collection, (4) data analysis, (5) conclusion and recommendation. The methods will be presented in the next section. The data was collected from the results of existing studies, and secondary data from relevant agencies.
The data used for the research consisted of the following:

- Gross Regional Domestic Product (GRDP) in Karawang regency from 2000-2010 in various sector, such as: Non-oil and gas sector, Oil and gas sector, Agriculture, Industrial, Commercial sector, Population
- map of road network, and terrestrial map of agriculture, industrial and agriculture

Model is formatted using linier model with three components of urban spatial component which are Commercial sector (C), industry sector (I), and agriculture sector (A). Mostly, commercial sector in the cities of Indonesia naturally concentrated in the downtown, whereas the cost of area in the downtown is relatively high. Otherwise, industry sector is located in the suburb that connected directly to the main road in order to distribute the industry products. Real estate area is spread inside the city.

### 4 DATA ANALYSIS

Frequently, the toll road construction has been related to change in land-use and mobility rate of the community structure. One can identify development of particular area by the increase of gross domestic regional product (GDRP) at any given time span. The standpoints for this progress in this research were agriculture, industry (other than oil and gas), trade, and population density. The secondary data that obtained processed by the SPSS 16.0 application software, giving the following results:

Summary of agricultural- and industrial-based attribute regression ($\Delta X_1$)

<table>
<thead>
<tr>
<th>Correlations</th>
<th>AGRICULTURE</th>
<th>INDUSTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pearson Correlation</td>
<td></td>
<td>0.962</td>
</tr>
<tr>
<td>Sig. (1-tailed)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>N</td>
<td>11</td>
<td>11</td>
</tr>
</tbody>
</table>

Variable correlation coefficient agricultural sector to industrial sector = 0.962, sig. (1-tailed) = 0.000 had interpretations as follows:

- Probability rate 0.000 was less than 0.05 complying with requirement. There was a significant correlation between agricultural and industrial sectors.
- Correlation coefficient of 0.962 was marked positive. The correlation was positive, in which the higher agricultural value, the higher industrial value.

Summary of agricultural-, industrial-, trade-, and population-based attribute regression

<table>
<thead>
<tr>
<th>Correlations</th>
<th>AGRICULTURE</th>
<th>INDUSTRY</th>
<th>TRADING</th>
<th>POPULATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pearson Correlation</td>
<td></td>
<td>0.919</td>
<td>0.972</td>
<td>0.972</td>
</tr>
<tr>
<td>Sig. (1-tailed)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>N</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td>11</td>
</tr>
</tbody>
</table>

The above table is a matrix of the variable correlation between agricultural, industrial, trade, and population density sectors. The N rate of each sector was 11 and the technique of analysis used was Pearson correlation. The above output led to the interpretation of the correlation that whether correlation existed, it followed the below requirements:

- If probability rate was less than 0.05, H0 was rejected, and a significant correlation existed.
• If correlation was marked negative, a positive (one-direction) correlation existed; the higher the score of the first variable, the higher the score of the second variable, vice versa.

• If correlation coefficient was marked negative, a negative direction (opposed-direction) correlation existed; the higher the score of the first variable, the lower the score of the second variable, vice versa.

Based on the above requirements one could enter the correlation interpretation. From the significance aspect, all of the correlation between both of three sectors had high correlation which is showing a significant correlation.

<table>
<thead>
<tr>
<th>Model</th>
<th>R</th>
<th>R Square</th>
<th>Adjusted R Square</th>
<th>Std. Error of the Estimate</th>
<th>R Square Change</th>
<th>F Change</th>
<th>df1</th>
<th>df2</th>
<th>Sig. F Change</th>
<th>Durbin-Watson</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>.947</td>
<td>.894</td>
<td>.878</td>
<td>0.047</td>
<td>0.947</td>
<td>41.752</td>
<td>3</td>
<td>7</td>
<td>0.000</td>
<td>1.948</td>
</tr>
</tbody>
</table>

The above table explained the percentage of the effect of the predictor variable on dependent variable. The determination coefficient was 0.947, so the independent variable effect on the change in the dependent variable was 94.7%. Therefore, agricultural sector was more affected by industrial, trade, and population sectors. It was generally understood because the correlation coefficient between variables showed a significance rate.

The F value was 41.758, significance rate was 0.000a, and H0 was rejected. Therefore, the variation of the independent variable was able to explain the independent variable. In other words, industrial, trade, and population sectors were able to predict the value of the agricultural sector.

<table>
<thead>
<tr>
<th>Model</th>
<th>Sum of Squares</th>
<th>df</th>
<th>Mean Square</th>
<th>F</th>
<th>Sig.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regression</td>
<td>2.93E+15</td>
<td>3</td>
<td>9.80E+14</td>
<td>41.752</td>
<td>.000</td>
</tr>
<tr>
<td>Residual</td>
<td>1.02E+15</td>
<td>7</td>
<td>2.33E+09</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>3.95E+15</td>
<td>10</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Column B at the constant was constant with 1.576E6, whereas value for industry was 0.160, trade was -0.182, and population was -0.128. It resulted in the following equation/model:

\[ Y = a + b1X1 + b2X2 + b3X3 \]
\[ = 1.576E6 + 0.160X1 - 0.182X2 - 0.218X3 \]

Where:

- \( Y \) = agricultural sector value ; \( X1 \) = industrial sector value ; \( X2 \) = trade sector value; \( X3 \) = population density sector value

Based on series of tests, it was found that the dominant effect of the land-use management went to agricultural and industrial sectors. When the agricultural area was replaced by industrial site, then the agricultural value changed along with the change in industrial value. In other words, the change in industrial sector would affect the value of agricultural sector. Furthermore, trade sector did not always use agricultural area for trade/commercial activities because it dealt with vary products and population density did not have any direct correlation to the increasing value of the agricultural sector.

Industrial fast growth needed more accessibility to leverage the service and goods distribution. Facilities were significantly required. To overcome the problems that potentially arose, roads were built to enable distributional trips and commodity delivery. However, arterial roads were not always capable of accommodating the traffic volume.
Therefore, a volume comparison had to be performed at the toll roads, as follows:

- If ADT of a given area was 1530.11 veh/hour, the comparative load for arterial and toll roads would be interpreted as follows:
  - Load on 4/2D-type highway road $V/C = 0, 49 \approx 0.5$
  - Service rate of the road was at C level, in which traffic flow as stable but the speed and movement of the vehicles were under control and speed was limited.

Hence, there was a great expectation that the toll road construction would lead to problem-solving other volume load on major road branches because this step was very important to promote goods and service delivery and distribution towards further development. Emerging activities alongside the toll roads were also under strict consideration.

5 CONCLUSIONS AND RECOMMENDATION

Based on the result of the data analysis, it is indicated that the dominant factors, which influence on land-use are agricultural sector and industrial sector. If the agricultural area replaced by the number of industries, then the value of agriculture will be affected directly proportional to the value of industry. In other words, the increment of the number of industries that replace agricultural area will be affecting the value of the agricultural factor.

The rapid growth of industry certainly requires high accessibility to distribute the industry product. Therefore, some facilities are entailed to support those industry activities. The toll road construction will be followed by other activities, such as industries, real estate, and many more. In consequence, land use arrangement and toll road accessibility is highly required.

These findings suggest several courses of action for future practice. The planning of land use should therefore be to design by local government for the long-term care of the area. Industry area, agriculture, trading, and real estate are supposed to be well-managed. Moreover, those areas should be equipped by the access road which connected to them. Well-planned of land use in the toll road could avoid the negative impacts in the future.

REFERENCES


PAPER TITLE
(90 Characters Max)
The Improvement of Pavement and Its Future Prospects in Taiwan

TRACK
Pavements & Materials

AUTHOR
(Capitalize Family Name)
Jaw, Shing-Hau

POSITION
Director

ORGANIZATION
Taiwan Directorate General of Highways, MOTC

COUNTRY
Taiwan (R.O.C.)

CO-AUTHOR(S)
(Capitalize Family Name)
Hwang, Sunn-Jer
Chen, Jyh-Lin

POSITION
Director
Section Chief

ORGANIZATION
Materials Testing Laboratory, DGH, MOTC

COUNTRY
Taiwan (R.O.C.)

E-MAIL
(for correspondence)
hwang@thb.gov.tw

KEYWORDS:
International Roughness Index (IRI)
Cumulative Percentage
Box Plot
Ride Quality

ABSTRACT:
Since 2007, the Taiwan Directorate General of Highways (DGH) has begun to use the International Roughness Index (IRI) to measure pavement roughness of all provincial highways. The annual measuring volume is about 9500 km under the calculation of a single traffic lane. The DGH has completed 7-cycle measurements till 2013, and the results have been faded back to the road maintenance management system. During these years, the pavement roughness has been considerably improved. From 2007 to 2013 the IRI mean value fell from the 4.82 m/km to 3.65 m/km, dropped significantly by 24%. Standard deviation gradually reduced from 1.57 m/km to 0.98 m/km. In 2009, we set 3 IRI targets--one for the short-term before 2009, another for the mid-term before 2011, and the other for the long-term before 2013. And, we have obtained 99-percent of our previously-set targets. With a view to more effectively utilizing the road maintenance budget, we, in 2014, classified roads into 5 groups, and set 5 different IRI targets respectively according to their different geographical areas and different designs of speeds. Using rolling management, we expect to more effectively use the test results, and achieve perfect and more efficient pavement maintenance; further, the DGH can provide the Taiwan’s public for enjoying better road environments.
The Improvement of Pavement and Its Future Prospects in Taiwan

Jaw, Shing-Hau¹ Hwang, Sunn-Jer² Chen, Jyh-Lin³

¹Director of Taiwan Directorate General of Highways, Taiwan (ROC)
²Director of Materials Testing Laboratory, Taiwan Directorate General of Highways, Taiwan (ROC)
³Section Chief of Materials Testing Laboratory, Taiwan Directorate General of Highways, Taiwan (ROC)

Email for correspondence: hwang@thb.gov.tw

1 PREFACE

There are national highways, provincial highways, city highways, county highways and country roads in Taiwan (Republic of China Highway Act 2013). In this paper, we emphatically study provincial highways used as communications between two or more counties, or municipalities (or provinces), or major political, economic and cultural centers. The total length of provincial highways is about 5,000 kilometers, and its paved area is about 90 square kilometers. Taiwan with the area of 36,000 square kilometers and with the population of 23,000,000 has the character of heavy traffic. The average daily traffic is about 11,000 passenger car unit (PCU) on provincial highways; and its maximum daily traffic is about 87,000 PCU. The peak hour volume (PHV) is at an average of 1,300 PCU; and its maximum is about 8,700 PCU (DGH website).

Since 2007, the Taiwan Directorate General of Highways (DGH) started to measure the pavement roughness of all provincial highways by employing the International Roughness Index (IRI). Basically, one testing cycle is implemented each year, by choosing a certain lane to do forward and back-ward (two-direction) tests. After the completion of tests, their analyses and statistics will be used in road maintenance and management.

2 INTRODUCTION OF TESTING INSTRUMENTS

Figure 1 is the photo of DGH’s pavement testing car. Testing instruments are put in the right and left instrumental boxes in front of the car (just next to bumper) where 2 laser displacement sensors are set. The laser displacement sensors are used to measure the height difference between emitting point and the pavement surface. Sampling frequency is 1250Hz. Following the longitudinal sampling Resolution≤25mm regulated by ASTM E950 class 1, we take longitudinal sampling resolution 11mm at the speed of 50km/hr; 22mm at the speed of 100km/hr. Two accelerometers are set in right and left instrumental boxes, establishing a reference baseline for calculating elevation changes of pavement (measuring scope, ±5g). One distance measurement instrument (DMI) is set on left rear tire, and can emit 2000 pulse signals per revolution. Because of the circumference of the above tire being 2125 mm, one signal is obtained by driving 1.1 mm. All the above obtained signals, which are transmitted to the computer server in the testing car, are further analyzed and calculated by the related software in a notebook computer. The repeatability and accuracy of the testing instruments which were approved by the Tjing Ling Industrial Research Institute, National Taiwan University, in 2013, meet the requirements of AASHTO R56.

![Figure 1. The photo of DGH’s pavement testing car](image-url)
3 TESTING RESULTS IN THE PAST YEARS

During 2007-2013, annual measured mileage of provincial highways shown in Table 1 was about 9,500 lane kilometers. Among them, a slight increase or decrease of length resulted from completion of new roads, not-yet recovered roads because of natural disasters. The mean value of IRI (m/km), standard deviation of IRI (m/km), Max. of IRI, Min. of IRI, cumulative percentage of IRI < 5m/km and cumulative percentage of IRI < 4m/km in the past some years are statistically shown in Table 2. As shown in Table 2, the mean value of IRI gradually decreased year after year. For example, we reduced the mean value of IRI from 4.82m/km in 2007 to 3.65m/km in 2013. The reduction percentage reaches 24%. The standard deviation of IRI was changed from 1.57m/km into 0.98m/km. It means that the distribution had been gradually concentrated. The maximum of IRI has been changed from 15.46m/km into 8.37m/km. The minimum of IRI has been changed from 1.55m/km into 0.92m/km. All these evidences show that the surface of pavement has become flatter and flatter. Figure 2 shows the distribution of IRI in the past years, but too many curve lines are difficult of being read. Therefore, we only choose 4-year cumulative curves which are left-upward. This means that the percentage of flat surface of pavement is becoming high. For example, the cumulative of percentage of IRI < 5m/km has been increasingly changed from 60.1% into 90.5% year after year; similarly, the cumulative of percentage of IRI < 4m/km has been increasingly changed from 34.8% into 67.1% year after year.

Table 1. Measurement mileage of provincial highways from 2007 to 2013

<table>
<thead>
<tr>
<th>Year</th>
<th>2007</th>
<th>2008</th>
<th>2009</th>
<th>2010</th>
<th>2011</th>
<th>2012</th>
<th>2013</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lane km</td>
<td>9406</td>
<td>9568</td>
<td>9124</td>
<td>9444</td>
<td>9566</td>
<td>9594</td>
<td>9627</td>
</tr>
</tbody>
</table>

Table 2. Statistics of IRI from 2007 to 2013

<table>
<thead>
<tr>
<th>Year</th>
<th>2007</th>
<th>2008</th>
<th>2009</th>
<th>2010</th>
<th>2011</th>
<th>2012</th>
<th>2013</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean Value of IRI (m/km)</td>
<td>4.82</td>
<td>4.76</td>
<td>4.32</td>
<td>4.31</td>
<td>4.03</td>
<td>4.08</td>
<td>3.65</td>
</tr>
<tr>
<td>Standard Deviation of IRI (m/km)</td>
<td>1.57</td>
<td>1.42</td>
<td>1.25</td>
<td>1.22</td>
<td>0.99</td>
<td>0.96</td>
<td>0.98</td>
</tr>
<tr>
<td>Max. of IRI (m/km)</td>
<td>15.46</td>
<td>13.75</td>
<td>13.10</td>
<td>11.45</td>
<td>9.92</td>
<td>8.33</td>
<td>8.37</td>
</tr>
<tr>
<td>Min. of IRI (m/km)</td>
<td>1.55</td>
<td>1.38</td>
<td>1.41</td>
<td>1.32</td>
<td>1.43</td>
<td>1.25</td>
<td>0.92</td>
</tr>
<tr>
<td>Cumulative Percentage of IRI &lt; 5 m/km</td>
<td>60.1%</td>
<td>61.3%</td>
<td>74.7%</td>
<td>75.2%</td>
<td>84.3%</td>
<td>83.6%</td>
<td>90.5%</td>
</tr>
<tr>
<td>Cumulative Percentage of IRI &lt; 4 m/km</td>
<td>34.8%</td>
<td>32.0%</td>
<td>44.9%</td>
<td>45.2%</td>
<td>52.6%</td>
<td>49.3%</td>
<td>67.1%</td>
</tr>
</tbody>
</table>

Figure 2. Cumulative distribution curves of IRI respectively in 2007, 2009, 2011 and 2013.

Figure 3 shows the distribution changes of IRI in the past years. Rectangular blocks with different colors represent different intervals of IRI. From the figure, we see each IRI interval is gradually upward year by year. For example, the part IRI<3m/km (green block) is enhanced from 9% into 27%, and the part IRI>6m/km (red block) is decreased from 21% into 2%. Apparently, we have made good use of the statistics of IRI to more effectively maintain roads. The testing results are useful and beneficial to the mechanism of road maintenance and management.
4 DISTRIBUTIONS OF IRI IN ACCORDANCE WITH DIFFERENT SPEED LIMITS

Provincial highways are scattered over all the island of Taiwan, composed of straight expressway and roads located respectively in plains, hills, mountains. For example, there is a provincial highway just with the length of 31.4 km in Taiwan, but its elevation is changed even from 1100m into 3275m – very great difference of elevations. Therefore, different grades of roads should be maintained by different standards of maintenance. According to different speed limits, we statistically analyse so as to understand the pavement roughness on the roads with different grades. In 2013, We tested 9627 lane km on provincial highways which were further divided into 5 groups--- (1) speed limit ≤ 35km/hr (at very special mountainous areas), (2) speed limit=40km/hr (mountainous areas), (3) speed limit=50km/hr (hilly areas), (4) speed limit≥60km/hr (plain areas), and (5) speed limits generally between 70 to 90km/hr (on most expressway). In Figure 4, the each tested length (lane km) of the above 5 groups is as follows: Speed limit≥60km/hr, 4366 lane km, 45% of total tested length; Speed limit=50km/hr, 2066 lane km, 22%; Expressway, 1153 lane km, 12%; Speed limit≤35km/hr, 1076 lane km, 11%; Speed limit=40km/hr, 966 lane km, 10%.

The mean values, standard deviation, maximum, minimum of IRI and in 68% scope under different speed limits are shown in Table 3. The box plots of analyzed IRI data of each group are drawn as Figure 5. The substantial blue frames mean (1) the upper boundary line representing mean value + 1 standard deviation and, (2) the lower boundary line representing mean value – 1 standard deviation. Vertical lines mean maximum (up-prolonged) and minimum (down-prolonged). From the above figure and table, we understand the mean value, maximum, and minimum are becoming smaller as the speed limits are becoming higher. In other words, higher speed limits consequently make better pavement roughness. Most of the low speed limits in mountainous area, pavement implementation and maintenance are more difficult. Standard deviations are becoming smaller as the speed limits are becoming higher. And, IRI data are more concentrated, and in narrower distribution.
Table 3. IRI statistical analysis of different speed limits

<table>
<thead>
<tr>
<th>Speed Limit Classification</th>
<th>IRI (m/km)</th>
<th>Mean Values</th>
<th>Standard Deviation</th>
<th>Max.</th>
<th>Min.</th>
<th>68% Scope</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤35km/hr</td>
<td>4.80</td>
<td>0.97</td>
<td>8.37</td>
<td>2.36</td>
<td>3.82~5.77</td>
<td></td>
</tr>
<tr>
<td>40km/hr</td>
<td>4.26</td>
<td>0.91</td>
<td>6.63</td>
<td>2.14</td>
<td>3.35~5.18</td>
<td></td>
</tr>
<tr>
<td>50km/hr</td>
<td>3.83</td>
<td>0.83</td>
<td>7.09</td>
<td>1.81</td>
<td>3.00~4.66</td>
<td></td>
</tr>
<tr>
<td>≥60km/hr</td>
<td>3.38</td>
<td>0.75</td>
<td>6.37</td>
<td>0.92</td>
<td>2.63~4.12</td>
<td></td>
</tr>
<tr>
<td>Expressway</td>
<td>2.80</td>
<td>0.71</td>
<td>5.24</td>
<td>1.13</td>
<td>2.09~3.51</td>
<td></td>
</tr>
<tr>
<td>All of The Provincial Highway</td>
<td>3.65</td>
<td>0.98</td>
<td>8.37</td>
<td>0.92</td>
<td>2.67~4.63</td>
<td></td>
</tr>
</tbody>
</table>

Figure 5. IRI box plot of different speed limits

5 COMPARISON OF PAVEMENT AMONG DIFFERENT COUNTRIES

We tried to do the pavement comparisons with those of other countries. After consulting the related information from many web sites, the American some state government information, including Washington (WSDOT 2013) · California (Caltrans 2011) · Florida (FDOT 2010) · New Jersey (NJDOT 2008), showed the entire state roads or only particularly concentrated in a certain area; in other words, roads are not classified into different grades, so we cannot take them to do comparison with those of Taiwan’s provincial highways. The only information we obtained is the pavement testing report by Virginia State in 2012 (VDOT 2012). Virginia State classified its roads into different grades to do statistics of IRI, and evaluated ride quality five grades from excellent to very poor. Their relative IRI are shown in Table 4. The IRI thresholds under the acceptable ride quality should be smaller than 2.21m/km on interstate & primary roads, and smaller than 3.47m/km on secondary roads. Because of VDOT web site stating that the limits of secondary roads are 45 mph for trucks and 55mph for other vehicles (VDOT website), we consider doing relative and matching comparison by citing (a) Taiwan’s provincial highways, including expressway, with speed limit≥70km/hr, 2863Lane Km tested in 2013, and (b) those of secondary roads in Virginia State, USA. We statistically obtain: the mean value of IRI 3.09m/km, standard deviation 0.76m/km, maximum 6.12m/km and minimum 0.92m/km. The comparison results are shown in Figure 6. Excellent Grade 1.2% and Good Grade 30.6% of Taiwan’s provincial highways are little worse than Excellent Grade 2.9% and Good Grade 33.5% of Virginia’s secondary roads. But, Poor Grade 4.8% of Taiwan’s is better than Poor Grade 10.7% of Virginia’s. If we cite IRI thresholds (IRI<3.47m/km ) as Virginia’s acceptable Ride Quality, 70% of Taiwan’s provincial highways are acceptable, and slightly better than 66% of Virginia’s secondary roads.

Table 4. Rating in ride quality on Virginia’s roads

<table>
<thead>
<tr>
<th>Ride Quality</th>
<th>IRI Rating (m/km)</th>
<th>Interstate &amp; Primary</th>
<th>Secondary Roads</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excellent</td>
<td>&lt;0.95</td>
<td>&lt;1.50</td>
<td></td>
</tr>
<tr>
<td>Good</td>
<td>0.95~1.57</td>
<td>1.50~2.67</td>
<td></td>
</tr>
<tr>
<td>Fair</td>
<td>1.58~2.20</td>
<td>2.68~3.46</td>
<td></td>
</tr>
<tr>
<td>Poor</td>
<td>2.21~3.15</td>
<td>3.47~4.41</td>
<td></td>
</tr>
<tr>
<td>Very Poor</td>
<td>≥3.16</td>
<td>≥4.42</td>
<td></td>
</tr>
</tbody>
</table>
Figure 6. Comparison of ride quality of pavement between Taiwan’s provincial highways and Virginia’s secondary roads

Additionally, we cite the report of Irish regional road network (Kieran & Brian 2012), and do the comparison between all Taiwan’s provincial highways and roads mentioned in the above Irish report. Taiwan’s IRI 3.65m/km is slightly better than Irish IRI 4.2m/km. Cumulative distributions are shown as Figure 7. Taiwan’s curve is more approaching S curve; and Irish curve is quadratic curve. The intersection of the two curves is at IRI 3.7 m/km. The Irish cumulative percentage on the left part of intersection is slightly higher than that of Taiwan; but, on the right part of intersection, Taiwan’s cumulative percentage is much higher than that of Irish Republic. From Figure 7, we simultaneously understand that the IRI distribution of Taiwan’s provincially highways is more concentrated than that of Irish Republic.

Figure 7. Comparison of IRI cumulative distribution between all Taiwan’s provincial highways and Irish regional roads

6 TARGET-REACHING AND FUTURE PROSPECTS IN TAIWAN

In 2008, the second year when we observed, we found IRI mean value changed from 4.83m/km of year 2007 into 4.76m/km of year 2008; and similarly, IRI>7m/km, changed from 91.4% into 93.0%. In other words, the effect was not apparent. With a view to letting IRI testing results be applied to road maintenance system and effectively improve the pavement roughness so as to offer the public with better quality of roads, we, DGH, in 2009, set short-term, middle-term and long-term programs to improve the pavement roughness. Before the end of 2009, the short-term target of all Taiwan’s provincial highways with IRI<7m/km is achieved except mountainous ones. Before the end of 2011, the middle-term target of all Taiwan’s provincial highways with IRI<6m/km is achieved. Before the end of 2013, the long-term target of all Taiwan’s provincial highways with IRI<5.5m/km is achieved. According our previously-set targets, we review our improvement program, excluding the statistics of mountainous roads (speed limit ≤35km/hr and speed limit=40km/hr). Through past some-year improvement, the results about the short-term target before 2009, the middle-term target before 2011 and the long-term target before 2013 are respectively shown in Table 5. Target-reaching
percentages are 99.2% for short term, 98.9% for middle-term and 98.8% for long-term. Totally speaking, there was only 1% which was not achieved.

Table 5. Target-reaching situation in short-term, middle-term and long-term improvement programs of pavement roughness

<table>
<thead>
<tr>
<th>Target Classification</th>
<th>Term Goal Achievement</th>
<th>The Intended Target</th>
<th>Total Lane km</th>
<th>Target-Reaching Lane km</th>
<th>Target-Reaching Percentages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Short-Term</td>
<td>The End of 2009</td>
<td>IRI&lt;7m/km</td>
<td>7314</td>
<td>7254</td>
<td>99.2%</td>
</tr>
<tr>
<td>Middle-Term</td>
<td>The End of 2011</td>
<td>IRI&lt;6m/km</td>
<td>7510</td>
<td>7429</td>
<td>98.9%</td>
</tr>
<tr>
<td>Long-Term</td>
<td>The End of 2013</td>
<td>IRI&lt;5.5m/km</td>
<td>7583</td>
<td>7493</td>
<td>98.8%</td>
</tr>
</tbody>
</table>

During the past 4 years, the DGH has positively injected huge budget into road maintenance so as to best improve the IRI values—USD 57 million in 2009, USD 69 million in 2010, USD 121 million in 2011 and USD 95 million in 2012 as shown in Figure 8. The upper one of the Figure 8 shows the changes of IRI values from 2009 to 2013. The lower one of the figure 8 shows the budgets from 2009 to 2012. On average, annual costs of pavement roughness maintenance for Taiwan’s provincial highways are about USD 88 million. And, it is great to obtain such good improvements. We find in the long run the injected costs can decrease the IRI values, but there is no absolute proportional relationship between injected costs and reduction degree of IRI values. This phenomenon maybe results from local typhoons and frequent earthquakes. For example, a newly constructed road may possibly completely destroyed by a typhoon and all maintenance costs in poured become void. If we set a 2-year observation period as shown in Table 7, the average reduction of IRI values is 0.29m/km from 2009 to 2011; the reduction percentage is 6.7% (by 0.29/4.32). The average reduction of IRI values is 0.38m/km from 2011 to 2013; the reduction percentage is 9.3% (by 0.38/4.03). During the observation period, we respectively poured USD 136 million and USD 216 million. And, the IRI reduction effect per million dollars was 0.049% and 0.043% respectively. So, the both reduction effects were very close. The lower IRI values they were, the lower reduction effects we got. At present, pavement roughness of Taiwan’s provincial highways is now faced with rapid improvement, so high IRI reduction effects were obtained. The ratio of IRI reduction value to maintenance costs will be diminished. By our continuous observation, we think the IRI values will become very stable.

Figure 8. The changes of average IRI values of Taiwan’s provincial highways, and their relationships with injected maintenance costs from 2009 to 2013

Table 7. The comparison between the changes of IRI values of Taiwan’s provincial highways and annual injected maintenance costs from 2009 to 2013.

<table>
<thead>
<tr>
<th>Year</th>
<th>2009</th>
<th>2011</th>
<th>2013</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean Value of IRI (m/km)</td>
<td>4.32</td>
<td>4.03</td>
<td>3.65</td>
</tr>
<tr>
<td>The 2-year reduction of IRI values (m/km)</td>
<td>0.29</td>
<td>0.38</td>
<td></td>
</tr>
<tr>
<td>The 2-year reduction percentage of IRI</td>
<td>6.7%</td>
<td>9.3%</td>
<td></td>
</tr>
<tr>
<td>2-years of investment in road maintenance funding</td>
<td>$136 million</td>
<td>$216 million</td>
<td></td>
</tr>
<tr>
<td>Each invested $1 million funding decline in the average IRI</td>
<td>0.049%</td>
<td>0.043%</td>
<td></td>
</tr>
</tbody>
</table>
The short-term and middle-term targets which we set last time have been reached. We hope the continuous improvement of pavement roughness will be obtained. In 2014, we set different targets toward different grades of roads; and simultaneously different maintenance thresholds are made so as to let targets be clear and more effective. Therefore, we in 2014 make 5-group regulations according to 5 different speed limits—speed limit $\leq 35$km/h, speed limit $= 40$km/h, speed limit $= 50$km/h, speed limit $\geq 60$km/h and higher speed limit applied on expressway. Using the statistical Figure 9, we approximately estimate the costs used to reach the target IRI values so that the budgets will be completely optimized. Generally speaking, mean value plus or minus some times of standard deviation is used to set maximum and minimum in quality control. Therefore, we preliminarily set (mean value) + (1 x standard deviation) as our maintenance threshold shown as in Table 8. From the view-point of normal distribution, there are 16% of Taiwan’s provincial highways which IRI are higher than the thresholds which we set. So, we focus our maintenance attention on the 16% of roads, and will review the maintenance thresholds according to the final testing results in 2014.

![Cumulative distribution curves under different speed limits](image)

**Figure 9.** IRI cumulative distribution curves under different speed limits

<table>
<thead>
<tr>
<th>Speed Limit Classification</th>
<th>$\leq 35$km/hr</th>
<th>$40$km/hr</th>
<th>$50$km/hr</th>
<th>$\geq 60$km/hr</th>
<th>Expressway</th>
</tr>
</thead>
<tbody>
<tr>
<td>IRI maintenance thresholds (m/km)</td>
<td>5.77</td>
<td>5.17</td>
<td>4.66</td>
<td>4.13</td>
<td>3.51</td>
</tr>
</tbody>
</table>

**Table 8.** Suggestive standards of IRI in roads’ maintenance

7 CONCLUSIONS

Since we, DGH, started to deal with testing the pavement roughness of Taiwan’s provincial highways, the related operations—planning, tests-executing, review of target-reaching, road maintenance and management mechanism fed back by testing results and the executing of pavement maintenance, have been implemented during the past 7 years. From PDCA cycle, the review of target-reaching and subsequent improvements has continuously made pavement roughness of Taiwan’s provincial highways better. Though the values of IRI are the same, comfortable feelings are different, resulting from different speed limits. We will set targets and review them according to different speed limits so as to let testing results be more widely used and let us be able to deliver more proper counter-proposals. Additionally, national resources will be more effectively implemented under the circumstances of our governmental tight financial situation.

IRI is not the sole factor in deciding the allocation of maintenance budgets or in enjoying maintenance superiority. As a matter of fact, there are more other factors, such as consideration of other pavements, traffic flux, population, economy and politics. Then, maintenance will be more precisely implemented. Because of the limitation of the writing length, we focus the research paper only on IRI.

Compared with those of other countries, the Taiwan’s pavement roughness has been close to the international standards. We, DGH, are looking forward to endeavoring to obtain continuous improvements.

8 REFERENCES

"Highway Act", 2013.07, Taiwan, the Republic of China
California department of transportation, Dec 2011, "2011 State of the Pavement Report"
State of Florida, June 2010, "Flexible Pavement Smoothness Acceptance Report International Roughness Index"
New Jersey department of transportation, August 2008, "Report to The Governor and The Legislature on New Jersey's roadway pavement system fiscal year 2007"
Virginia department of transportation, November 2012, "State of the Pavement 2012"
Dr. Kieran Feighan, Brian Mulry, Nov. 2012 "Assessing the condition of the Irish regional road network 2011/2012", Engineers Ireland
Laboratory Experimentation of Bituminous Foam mix under Humid Curing Condition

<table>
<thead>
<tr>
<th>AUTHOR (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roslinah ABDUL KARIM</td>
<td>Engineer</td>
<td>Department of Roads, PWD</td>
<td>Brunei Darussalam</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CO-AUTHOR(S) (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Andrew DAWSON</td>
<td>Associate Professor</td>
<td>The University of Nottingham</td>
<td>United Kingdom</td>
</tr>
</tbody>
</table>

E-MAIL (for correspondence) evxrh1@nottingham.ac.uk

evxrh1@gmail.com

KEYWORDS:
Bituminous Foam Mix, Cement, Stiffness Modulus, Humid, Moisture

ABSTRACT:

Brunei Darussalam is a small country without any hard aggregate with which to make conventional pavement materials. Therefore, at present, aggregate must be imported at high cost. Local alluvial sandstone aggregates are available but these do not meet conventional performance requirements for unbound pavement materials. To attempt to make beneficial use of such aggregate in road construction in Brunei Darussalam the use of foamed bitumen mixes based on sandstone aggregates (‘sandstone foam mix’) was investigated as a possible base course material.

Given the humid tropical climate that prevails in Brunei Darussalam, the pavement endures the environmental effect of high temperature, high humidity and significant wetting by the frequent rainfalls. This paper focuses on the evaluation of the stiffness performance of the foam mix with which, often, poor performance resulted from humid curing. In a high humidity environment, the process of curing or moisture loss was slower than in the oven drying condition. Test results show the benefits of low dosages of cement additive in humid-cured sandstone foam mixes. During the early days of curing, the foam mix with no cement only lost the moisture to about 10% of the Optimum Moisture Content (OMC) of raw aggregates after about ten days, thus the stiffness of the mix could only reach about 1000MPa. In comparison, the ones with cement were able to reach about three times the stiffness modulus of the uncememented foam mixes, about 3750MPa, within a short period of four days but somehow retained moisture, 30% of OMC. The uncememented mixes could only reach the same stiffness modulus value as the one with cement when their moisture condition was dried to 0% of the OMC which would be impractical for almost all of Brunei Darussalam’s climate conditions.

It is shown that the cement not only aids the foam mix curing process by its hydration process but forms calcium silicate hydrate crystals that eventually help to fill any voids producing a much denser structure than the foam mix alone. The addition of cement is concluded to be essential to more rapidly develop the desirable performance of foam mix, particularly in humid curing conditions. On the basis of this study, recommendations for the use of foam mix asphalts in humid tropical climates are presented.
Laboratory Experimentation of Bituminous Foam Mix under Humid Curing Condition

Roslinah Abdul Karim\textsuperscript{1}, Andrew R. Dawson\textsuperscript{2}

\textsuperscript{1}Research Student, The University of Nottingham, United Kingdom, evxr1@nottingham.ac.uk
\textsuperscript{2}Associate Professor, The University of Nottingham, United Kingdom, andrew.dawson@nottingham.ac.uk

1 INTRODUCTION

Brunei Darussalam is located in South East Asia on the north west of Borneo Island, sandwiched between two East Malaysian countries, Sabah and Sarawak. Its road infrastructure has been continuously developed in the nation resulting in the demand for more resources, particularly aggregates. It is however a small country without any hard aggregate with which to make conventional pavement materials. Therefore, at present, aggregate must be imported at high cost. Local alluvial sandstone aggregates are available but these do not meet conventional performance requirements for pavement materials. To attempt to make beneficial use of such aggregate in road construction in Brunei Darussalam the use of foamed bitumen mixes based on sandstone aggregates (‘sandstone foam mix’) was investigated under specific curing condition that closely simulated the local climatic conditions.

2 REVIEW OF CURING STRATEGIES

Curing is the process of water loss in mixes to gain strength which, in practical situations, depends on local climate conditions. Most guidelines on use of foam mix (Asphalt Academy, 2009) were made without specifying any restrictions toward local condition or climates. Curing can be fairly rapid in favourable weather conditions such as where there is a hot temperature and dry environment. A short curing period presents an ideal situation as the newly constructed or rehabilitated road could be almost immediately opened to traffic. Brunei Darussalam, being located near to the equator, has no pronounced dry or wet season where one can assure curing would be at its most favourable. The climate of Brunei Darussalam is characterised by a tropical equatorial climate, which is notable for its uniformity with an average temperature of 30°C throughout the year. The temperature may reach 35°C to 40°C in hot afternoons and fall to 22°C in cool nights. The mean daily humidity value is about 80% with an average maximum reaching up to 97%.

The constant high humidity would increase the curing period, slowing the development of the best properties of the foam mix and thus, delaying the road opening. Inadequate curing would lead to a lack of early strength, due to the presence of moisture, causing premature pavement distress. In a moist foam mix the bonding has not fully developed to gain the ideal strength (Fu et al, 2009). Hence, a short research study was undertaken to investigate the development of stiffness modulus of foam mixes with the reduction of moisture under humid curing.

3 MATERIALS

3.1 Aggregates

Sandstones are the common rock that can be found in Brunei Darussalam. However for logistic reason, it was decided to work with aggregate from the UK with qualities very similar to those typical Brunei’s sandstones. The aggregates were sourced from the Craig Yr Hessg quarry in Swansea and originated from the Pennant formation. They were separated into size fractions (20mm, 14mm, 10mm, 5mm and dust). Since the utilisation of the sandstone aggregates in the UK is mainly for road surfacing works, the actual available nominal aggregate size is 14mm and the maximum size is 20mm, whereas the target gradation was in accordance with the typical grading obtained from a local quarry company in Brunei Darussalam, New Temburong Quarry (NTQ). The target gradation is shown in Figure 1 which lies at the mid-point of the Brunei Darussalam’s GS1 specified road base gradation envelope (CPRU, 1998) and runs roughly at the lower boundary of the foamed bitumen grading envelope (Asphalt Academy, 2009).
Figure 1 Target gradation compared with the GS1 road base gradation envelope and foamed bitumen material's gradation envelope

Figure 2 Shifted gradations starting at new particle size of 20mm to which the actual gradation was fitted

Therefore, this study chose to make adjustment to the particle size distribution curve by shifting it so as to allow for available raw U.K sandstone aggregates of maximum sieve size of 20mm instead of 37.5mm (or 50mm). A shift factor was employed to calculate the new consecutive sieve sizes for the new shifted particle size distribution curve. However, the percentage passing at each of the new sieve size was the same value at the previous consecutive sieve sizes as shown in Figure 2. The new grading would be the actual grading which was then consistently obtained by fractioning the materials received and reconstituting them accurately according to mass as shown in Table 1.
Table 1 Proportion of actual aggregate gradation used in the study

<table>
<thead>
<tr>
<th>BS Sieve Size (mm)</th>
<th>Percentage Retained (%)</th>
<th>Combined % Passing (Designed Gradation)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>20mm</td>
<td>14mm</td>
</tr>
<tr>
<td>37.5</td>
<td>0.00</td>
<td>0.96</td>
</tr>
<tr>
<td>20.0</td>
<td>0.00</td>
<td>0.96</td>
</tr>
<tr>
<td>14.0</td>
<td>0.00</td>
<td>14.55</td>
</tr>
<tr>
<td>10.0</td>
<td>0.00</td>
<td>2.81</td>
</tr>
<tr>
<td>5.0</td>
<td>0.00</td>
<td>0.52</td>
</tr>
<tr>
<td>2.36</td>
<td>0.00</td>
<td>0.01</td>
</tr>
<tr>
<td>0.425</td>
<td>0.00</td>
<td>0.06</td>
</tr>
<tr>
<td>0.075</td>
<td>0.00</td>
<td>0.07</td>
</tr>
<tr>
<td>Pan</td>
<td>0.11</td>
<td>0.07</td>
</tr>
<tr>
<td>Grading Proportion %</td>
<td>19</td>
<td>19</td>
</tr>
</tbody>
</table>

3.2 Bitumen

Foamed bitumen is produced by expanding the neat bitumen by heating it, at a temperature ranging normally from 150°C to 180°C, and injecting a small amount of ambient water by an air pressure into a stream of hot bitumen in an expansion chamber. The water turns to steam which forms as bubbles in the bitumen, those these collapse as the bitumen-steam mix cools. Bitumen 90pen grade was selected to use for the foaming process throughout this study and it was supplied by Total Bitumen Company, U.K.

There are two main parameters to be determined from the foaming process which indicate the optimum characteristics of foamed bitumen: expansion ratio and half-life. Expansion ratio is defined as the ratio of expanded and original volume of bitumen. Half-life is the time taken from the maximum expanded volume of bitumen to its half collapsed volume.

3.3 Cement

Cement has the ability to accelerate the curing process in cases where early strength in required and reduce the moisture sensitivity when a foam mix alone cannot perform optimally (Jitareekul, 2009 and Fu, 2010). It is a group of hydraulic binders that reacts with water to harden and solidified hence, it may become less sensitive to water. Therefore, it was selected as a co-treatment agent in this study. The common type of cement is anOrdinary Portland cement (OPC) but many of its type being sold in the U.K’s market contain admixtures thus, in order to avoid their effects, the study used CEM1 type which is a pure cement. CEM1 has high index strength of 52.5kN/mm².

4 SAMPLE PREPARATION AND CURING CONDITIONS MATERIALS

4.1 Foamed Bitumen Characteristics

As mentioned in the earlier section, the optimum foamed bitumen characteristic is defined by two main parameters: Expansion Ratio and Half-Life. These two parameters were determined at different sets of bitumen temperature and foaming water contents (FWC). The results were plotted in a graph to indicate the optimum value of expansion ratio and half-life of the foamed bitumen.
The foamed bitumen was produced with the WLB10, laboratory foamed bitumen rig, from the company Wirtgen. The foamed bitumen was found to produce its optimum characteristics at a half-life of 8s and an expansion ratio of 12 times when the temperature of base bitumen was in the range of 160°C to 170°C and when 2% foaming water content (i.e 9 litre/hour) was added.

4.2 Optimum Moisture Content (OMC) of Untreated Aggregates

OMC is an important parameter to determine the mixing moisture content (MMC) for the mix design of foam mix. It must be determined relative to the compaction method used, and is based on the untreated raw aggregates. A gyratory compactor was selected to be the main compaction method due to its availability and easy operation. However, the test followed the standard compaction test procedures and calculations (BSI, 1990).

4.3 Mixing Moisture Content (MMC)

The MMC is the added water content to a mass of dried aggregates in a mixer prior to mixing them with foamed bitumen. The recommended MMC for foam mix is 70% of OMC (Wirtgen, 2004). Separate experiments, not reported here, confirm the suitability of 70% of OMC (as opposed to 60% or 80%) for the sandstone aggregate studies. To minimize the drying effect, the loose was kept in a closed container at all times until being compacted.

4.4 Compaction

All test specimens were fabricated using a gyratory compactor (BS EN 12697, 2001), set at 600kPa vertical pressure at an angle of gyration of 1.25° (see Figure 3). The compaction was terminated after 100 numbers of gyrations. Immediately after the gyratory compaction process, all cylindrical specimens were left in the mould for 24 hours to gain cohesion and conditioned at temperature of 30°C before they were extruded to undergo the selected curing conditions as explained in the following sections. The mass of specimen was weighed before undergoing the curing conditions. Any moisture movement was assessed based on weighing the mass of specimens prior to the ITSM test and when necessary.

4.5 Determination of Testing Temperature

In order to closely simulate the Brunei Darussalam’s local field condition, (see Section 1), a temperature of 30°C was selected as an ambient temperature hence, all aggregate samples and compaction moulds were conditioned at a temperature of 30°C prior to use. It was also the test temperature for all specimens.

4.6 Curing Conditions

It is essential to evaluate the properties of foam mixes based on local field conditions where they are applied. Both temperature and humidity (or water submersion) can be important as these conditions will affect cement hardening and ‘breaking’ of water from within bitumen. However, it is not possible to fully replicate the real field condition in a standard laboratory and it would be a difficult task. Therefore, three laboratory curing procedures that represent three distinct field conditions experience by the road pavement in Brunei Darussalam were employed.
4.6.1 Dry and Wet Condition

A dry condition is considered to represent a long-term cured condition (Ruckel et al, 1982). In this study, dry cured specimens were placed in the oven for 3 days at 40°C immediately after they were extruded from the compaction mould.

4.6.2 Wet Condition

A wet condition represents the extreme worst condition. The test specimens were soaked in water at temperature 30°C for 24 hours prior to testing.

4.6.3 Humid Condition

Most of simulated field conditions being studied do not appear to be representative of Brunei Darussalam’s applications. The humid curing condition was replicated in the laboratory using a closed plastic container that was partially filled with water to create a humid environment above water. The water was heated using a fish tank heater to set and maintain a temperature of 30°C as can be illustrated in Figure 4. Test specimens were not allowed to be in contact with moisture, not even from the condensation effects hence, they were placed in a covered container open to air passage and inside the closed outer container. Although it is not possible to simulate the real field condition in laboratory, the results would show the development of stiffness modulus under the humid curing times, whether short or long term.

![Figure 4 Schematic illustration of a laboratory humid curing tank](image)

5 METHODOLOGY

5.1 Indirect Tensile Stiffness Modulus (ITSM)

Stiffness modulus, $S_m$, is a measure of load-spreading ability of the bituminous layers. It controls the level of traffic induced tensile strains at the underside of the roadside which are responsible for fatigue cracking together with the compressive strains induced in the sub-grade that can lead to permanent deformation. It can be measured by an ITSM test which is a non-destructive and defined by BS DD 213 (1993) and is defined as:

$$S_m = \frac{P(0.273 + \nu)}{\Delta h \cdot t}$$

where
- $\nu$ = Poisson’s ratio, 0.35
- $P$ = Peak load applied (N)
- $S_m$ = Stiffness Modulus (MPa)
- $\Delta h$ = horizontal deformation (mm)
- $t$ = Specimen’s Thickness (mm)

The test was deformation controlled with the test specimen exposed to repeated vertical loads across its diameter that induced horizontal deformation on the central diameter. The deformation was measured by external Linear Variable Transducers (LVDTs), which lined up with the specimen’s diameter (as shown in Figure 5). A value is chosen to ensure that sufficient signal amplitudes are obtained from the transducers in order to produce consistent results. The value was selected as 3 micrometres in this test due to the weak nature of the foam mix (Oluwaseyi, 2010).
The magnitude of the applied force is adjusted by the system during the first five conditioning pulses to achieve the specified target horizontal deformation.

5.2 Determination of Optimum Binder Content (OBC)

In this paper, the dry and wet curing conditions were only used for the purpose of the determination of optimum foamed bitumen binder contents. The optimum binder content was determined based on the Indirect Tensile Stiffness Modulus (ITSM) test results. The selected gradation was mixed at different percentage of foamed bitumen, 2%, 3%, 4% and 4.5% thus, four replicates of foam mixes were manufactured for each of the foamed bitumen contents.

All test specimens were kept in the compaction mould for 24 hours at a temperature of 30°C to gain cohesion before they were extruded to undergo the dry and wet curing as described in section 4.6. After the dry curing, the test specimens were subjected to ITSM test to determine the dry optimum binder content (Figure 6). Due to the limited material resources, the same tested dry-cured specimens were conditioned in a water bath at 30°C for 24 hours then followed by the ITSM test to determine the wet optimum binder content (Figure 7). For reference the much greater moduli obtained after dry curing and the different optimum are illustrated in Figure 5.

5.3 Experiment Design

The optimum foamed bitumen binder had been selected as 4% (by mass of dry aggregates) following the results of Indirect Tensile Stiffness Modulus (ITSM) tests performed following wet curing (see Figure 7). The research study included a foam-only mix and one with the addition of 1% cement (by mass of dry aggregates) to compare their performance during the humid curing. This proportion of cement was based on a preliminary study (Abdul Karim,
2012) in which 0, 1 and 2% by mass of cement as added to the foam-mix. A large increase in stiffness was observed for both rates of addition, but brittle concrete-like response was observed at the higher cement addition rate, particularly when the foaming rate was low. Thus, to maintain the desired response, typical of a flexible pavement layer, the addition of cement at 1% by mass of the foam mix was adopted for further study.

6 TEST RESULTS AND DISCUSSIONS

6.1 Development of Stiffness Modulus over the Humid Curing Period

This section presents the development of stiffness modulus during a humid curing period. The results are presented into two sections: a short and long term humid curing condition. The early strength of road pavements are critical as road engineers commonly require roads to be open to traffic almost immediately after completion in order to cut down on traffic delays. Early stage development of foam mix is important to indicate the period of which the road base can be overlaid with the top surface of hot asphalt mix layer.

The short term response would predict the effects of humid curing on the development of stiffness modulus in the early days whereas the long term period would assist in the study of the development of stiffness modulus and moisture in foam mixes until they reach an equilibrium state.

6.1.1 Short Term Humid Curing Period

Figure 8 shows the development of stiffness modulus of foam mixes over a period of six weeks. The 4FB1C mix gained strength to about 3000MPa on the 4th day, which is about three times the stiffness of the 4FB mix, 1000MPa, which only gained this low value of strength after a week. An attempt was made to test 4FB on the 4th day, with only one replicate being tested and this gave a result of about 400MPa. Because of the weak result, the ITSM test on other replicates was discontinued. It was observed that the specimens were still in moist condition hence, it was decided to keep the test specimens for more days so that there was less risk of being severely damaged by the handling.

![Figure 8 Stiffness modulus of foam mixes over a month of humid curing](image)

After about six weeks, the stiffness modulus of 4FB mix only reached about 2500MPa. In contrast, the 4FB1C mix achieved an average stiffness of about 6500MPa, about twice its first ITSM test value on the 4th day of humid curing. The high stiffness modulus value of the 4FB1C mix demonstrates its potential application in a tropical humid climate.

The dry-cured foam mix with no cement reached stiffness modulus value for about 2000MPa however, the humid-cured foam mix with no cement could only reach the same stiffness modulus after one month. It indicates that the humid environment slowed the curing process of foam mix that may cause it to be prone to premature damage.
6.1.2 Long Term Humid Curing

Figure 9 presents a graph showing the development of stiffness modulus of foam mixes, 4FB and 4FB1C, when subjected to long term humid curing. The early stiffness value of the 4FB mix is 40 to 50% of the stiffness value of the 4FB1C mix at the same age. Stiffness of 4FB increases over time until, after four months, the curve is almost flat with a maximum average of approximately 3500MPa. In contrast, the stiffness modulus value of the 4FB1C mix could reach a maximum average of almost 8000MPa. However, its stiffness modulus eventually decreases somewhat, perhaps caused by the presence of micro-cracks or by cement paste carbonation.

Due to the limited material resources, the same test specimens were used for the ITSM tests throughout the investigation. Repeated ITSM tests might have presented differing aggregate orientations in the mixtures to the load platens, but this would lead to apparently random variation from test to test and this is not evidence in Figure 7. Cumulative damage to brittle cement bonds seems a more likely mechanism for mix 4FB1C, especially as no similar reduction is seen in 4FB mix.

After about a year of humid curing, the stiffness modulus of 4FB1C specimen approximately doubles relative to foam mix without cement, 4FB. The stiffness modulus of both foam mixes was even higher than their dry cured foam mixes. Therefore, this evident does not support the indication that the dry curing represents a long term site curing.

![Figure 9 Development of Stiffness Modulus under a Long Humid Curing](image)

6.2 Development of Stiffness Modulus with the Reduction in Moisture

The development of stiffness modulus with the reduction of moisture content can be seen in Figure 10. The results of dry cured foam mixes were also plotted in order to compare their stiffness modulus values to the humid cured foam mixes at reduced moisture content (OMC).
After 1 day kept in the mould at 30°C and 3 days of dry curing at 40°C, the foam mixes without cement, dry cured 4FB mix, had a moisture content reduced to about 5% of OMC. The dry cured 4FB mix had then gained a value of stiffness modulus between 2000MPa and 3000MPa, higher than the humid cured 4FB, which had a value of approximately 1500MPa only after 10 days. In order to reach the same stiffness as the dry cured 4FB, the humid cured 4FB had to continue to lose its moisture to almost 0% of OMC.

In contrast, the dry cured 4FB1C reached a stiffness modulus of about 5000MPa after the initial 4 days as compared to the humid cured 4FB1C that had then only reached between 3000MPa and 4000MPa. The dry-cured 4FB1C could reduce the OMC to about 25% within 4 days of dry curing, 5% less than the humid cured 4FB1C mix. The dry cured, 4-day value of 5000MPa could be achieved by the humid cured 4FB1C when its moisture reduced to about 15% of OMC.

The negative OMC’s values experienced by the humid-cured 4FB mix, plotted in the graph, indicates that there was a loss in the mass due to the disintegration of fines in the mix. Hence, the ultimate stiffness modulus reached by the humid-cured 4FB mix was approximately between 3000MPa and 4000MPa at 0% of OMC. This same value was reached by the 4FB1C mix after its 4th day of humid curing.

6.3 Moisture Movements in Humid Cured Foam Mixes

Figure 11 shows a plot of the moisture of foam mixes (assuming loss of mass indicates a loss of moisture) in terms of the percentage of OMC against curing period (in days). The tests show that the initial moisture content of humid cured 4FB mix, 70% of OMC, which was also the MMC of the foam mixes during mixing and compaction, reduced approximately 60%, 13% of OMC over a week of humid curing at 30°C. In contrast, the MMC of humid cured 4FB1C mix, 70% of OMC, reduced to about 20% of OMC hence it lost less moisture than 4FB mix.

Figure 10 Stiffness Modulus of Foam Mixes with the Reduction in OMC

Figure 11 Reduction of Moisture Content over the Curing Period
At an early age, the observed trends show similarity for both the 4FB and the 4FB1C mixes. The inclusion of cement did not seem to significantly affect the loss of mass. Therefore, the loss of water in the 4FB mix appears to be matched by an equal loss of moisture to the atmosphere plus an unquantified amount that has been reacted with the cement during the hydration process.

After one week, the 4FB mix lost about 60% from the initial moisture (MMC), 70% of OMC, more than the 4FB1C mix which lost to 22% of OMC. After about a month, the 4FB continued to lose moisture until it reached an almost horizontal line with maximum moisture lost being 0%. The negative values indicated that the 4FB mix experienced disintegration of fines. In contrast, for 4FB1C, the moisture lost stabilised at just above 0%.

When the moisture contents of the foam mixes reduced to about 20% of OMC, the 4FB mix continued to lose apparent moisture at a slow rate but clearly more than the 4FB1C mix until the moisture content reduced to about 0% of OMC, whereas the cemented mix lost less moisture. This is probably because the 4FB1C mix contains less, available water, due to the mass removed into hydration products.

6.4 Relationship between Stiffness Modulus and Moisture Contents

The graph in Figure 12 presents the experimental results of the stiffness modulus of 4FB and 4FB1C with the increasing loss in moisture under accelerated humid curing for approximately one year. The stiffness modulus generally increased with increasing moisture loss, as indicated by the previous graphs.

![Figure 12](image)

**Figure 12 Relationship between the Stiffness Modulus of Foam Mixes and Apparent Moisture Lost in Humid Curing**

For 4FB, the stiffness modulus reached approximately 1000MPa when the moisture loss was about 2.5%. As the moisture loss increased, the results became more scattered but somehow, the 4FB mix only lost a maximum of about 4.5% resulting in a stiffness modulus of 4000MPa. With the inclusion of cement, the 4FB1C gained a higher stiffness than the 4FB at all comparable moisture loss points.

7 DISCUSSION & CONCLUSIONS

Following the work described in this paper, it can be deduced that the stiffness characteristics of humid cured foam mixes are as follows:

Only a small amount of cement is required to assist the foam mixes in gaining early strength in order to reduce the risk of premature damage due to moisture and traffic loading.

The uncemented foam mix has a longer curing period than the cement treated foam mix and the stiffness modulus remains significantly less at all ages. It only rose to about 1000MPa after a week. The low stiffness modulus of the 4FB mix indicates that it has a high risk of moisture damage particularly in the event of rainfall during curing.
The stiffness modulus of the cement treated foam mix gained about 3000MPa by the 4th day of the humid curing period. The finding indicates that a short curing period of such a mix could allow for early trafficking.

After over 2 months of humid curing, the 4FB mix experienced disintegration, which was indicated by, apparently, negative values of the moisture content, calculated on the basis of loss in mass of the specimen. In contrast, the reduction of moisture in the 4FB1C mix reached a plateau, above 0% of OMC. This indicates that the 4FB1C mix used up the available water to form hydrated crystals that densified the microstructure.

Although the cement-treated foam mix reached a maximum stiffness modulus of 8000MPa, it eventually experienced reduction in the stiffness modulus after about 8 months perhaps due to micro-cracking or carbonation of the cement products.

Generally, the stiffness modulus of bituminous foam mix increases as the moisture lost increased. However, the results became variable once more than 3% moisture had been lost.

The work presented here has concentrated on the benefits to be achieved by adding cement to a foam-mix in order to benefit from humid curing when, otherwise, humid conditions might limit development of mechanical competency, or might cause deterioration of competency previously gained. It is beyond the scope of this paper to detail all the possible drawbacks or limitations of this type of treatment. However, the work performed earlier (Abdul Karim et al, 2012) indicated that higher rates of cement addition, especially to mixture with modest rates of foam addition, might lead to brittle mixes prone to cracking. Thus a careful study of the optimum balance between the two additions to the raw aggregate is recommended. Furthermore, no study on longer term issues such as cement-mortar carbonation has been considered. If the primary activity of the cement is to draw water from the bituminous phase and from bitumen-aggregate interfaces (which seems likely to be the case for 1% cement addition as it delivers a mix which is still fully flexible in behavior) then long-term damage mechanisms seem more likely to be mediated by the bitumen-stone interaction than by alterations to the cement-hydrates.

8 ACKNOWLEDGEMENTS

The authors would like to acknowledge the companies, Hanson Aggregates for supplying the aggregates sourced from Craig YrHessg Quarry, TOTAL Bitumen for providing bitumen and also the support of the laboratory technicians throughout the duration of the research study.

9 REFERENCES


Jitareekul P., (2009). An Investigation into Cold-in-Place Recycling of Asphalt Pavements, PhD Thesis The University of Nottingham, United Kingdom


**PAPER TITLE**
Monitoring of Structural Behavior of Corrugated Steel Plate Underpass during Construction

<table>
<thead>
<tr>
<th>TRACK</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>AUTHOR (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kyungsuk KIM</td>
<td>Chief Researcher</td>
<td>Korea Expressway Corporation</td>
<td>South Korea</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CO-AUTHOR(S) (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sangrae LEE</td>
<td>Researcher</td>
<td>Korea Expressway Corporation</td>
<td>South Korea</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>E-MAIL (for correspondence)</th>
</tr>
</thead>
</table>
Kskim2k4@ex.co.kr

**KEYWORDS:**
Corrugated Steel Plate Underpass, Structural Behavior, Monitoring, Vehicle Load Test

**ABSTRACT:**
The design and construction of the corrugated steel plate underpass should be very cautious because the structure can be subjected to excessive bending during backfill or under live-load when the soil cover is shallow resulting in excessive deformation or collapse. In this study, to verify the design calculation, a semi-circle arch underpass was instrumented and its structural behavior was monitored during backfill, and vehicle load test was conducted after the soil cover reached 0.6m and 1.0m. The result showed that maximum thrust of 198kN/m was measured at the shoulder location and maximum moment of 11.6kN-m/m was measured at the top and the shoulder location when the backfill reached the top of the structure. As the measured thrust was 1.52 times larger than the calculated thrust of 130kN/m and the moment was 2.5 times larger than the calculated moment of 9.74kN-m/m, it was concluded that the calculation to the code could underestimate the actual stress level developed in the structure during construction. When the measured value were compared with the plastic thrust and moment, the measured thrust was only 14.4% of plastic thrust and the moment was 78.4% of the plastic moment. The result implied that the reliable moment calculation during construction was important for structure stability.
Monitoring of Structural Behaviour of Corrugated Steel Plate Underpass during Construction

Dr. Kyungsuk Kim¹, Sangrae Lee¹

¹Korea Expressway Corporation, Hwaseong-si, Gyeonggi-do, KOREA
Email for correspondence: kskim2k4@ex.co.kr

1 INTRODUCTION

Since the corrugated steel plate structure was first introduced in 1997 as an underpass for Expressway in Korea, it has gained wide popularity in road construction. Although corrugated steel plate underpass has various advantages such as short construction period and low cost over concrete underpass, the design and construction of the structure should be very cautious because the structure can be subjected to excessive bending during backfill or under live-load when the soil cover is shallow, sometimes resulting in excessive deformation or total collapse of the structure.

In current design method suggested by CHBDC, axial thrust is examined based on ring compression theory and moment based on the semi-empirical method suggested by Duncan(1979). However, the behavior and stability of structure is highly dependent of construction sequence and depth of soil cover. In this study, to verify the design calculation, a corrugated steel plate underpass of semi-circle arch shape was instrumented with strain gages and its structural behavior was monitored during backfill, and vehicle load test was conducted after the soil cover reached 0.6m and 1.0m. The thrust and moment developed in the structure during backfill and load test were compared with the calculated according to the design method suggested by CHBDC( Canadian Highway Bridge Design Code).

2 MONITORING AND VEHICLE LOAD TEST SCHEME

The structure is semi-circle arch shape underpass of 7.5m span, 3.75m rise and 29m long upon the concrete footing. Soil cover over the structure is about 1.2m. Total 9 points were instrumented with strain gages with automatic temperature compensation along the inner surface of the plate. At each point, three strain gages at the crest, valley and neutral line respectively were used for calculation of mobilized moment. Figure 1 shows the cross-section and the instrumentation points.

![Figure 1. Structure Dimension and Location of instrumentation](image)

Table 1. Material Spec. of Corrugated Steel Plate

<table>
<thead>
<tr>
<th>Plate thickness (mm)</th>
<th>corrugation (mm)</th>
<th>Area of section (mm²/mm)</th>
<th>Section modulus (mm³/mm)</th>
<th>2nd moment of inertia (mm⁴/mm)</th>
<th>Yield strength $f_y$(MPa)</th>
<th>Ultimate strength $f_u$(MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.5</td>
<td>150×50</td>
<td>5.588</td>
<td>60.49</td>
<td>1648.4</td>
<td>245</td>
<td>400</td>
</tr>
</tbody>
</table>

During backfill operation, strains were measured at each backfill layer until the backfill reached to the soil depth of 0.3m on top of the structure. After the soil depth on the structure reached 0.6m and 1.0m, vehicle load test was conducted.
performed using the truck of 419kN total weight with 4-axles. In the test, different truck speed of 5, 10, 20, 40 km/hour was tried.

Axial thrust \( T \) and moment \( M \) mobilized in the structure were calculated using the equation (1) in which strains measured at 3 points was assumed to vary linearly as shown in Figure 2. (Beal, 1982).

\[
T = \left( \frac{1}{4} \sigma_1 + \frac{1}{2} \sigma_2 + \frac{1}{4} \sigma_3 \right) \cdot A
\]

\[
M = \frac{1}{2} (\sigma_1 - \sigma_2) \cdot Z
\]

Figure 2. Calculation Method of Thrust and Moment

3 MONITORING AND TEST RESULT

Axial thrust and moment during the backfill operation were shown in Figure 3. Axial thrust increases as backfill height increase and reaches maximum of about 198kN/m at the crest location. Normally, higher axial thrust was developed at the top portion of the structure. Moment also increases as backfill height increases and reaches maximum of 10.6kN-m/m at the crest location. However, after backfill reaches on top of the structure, moment begin to decrease as the soil cover works as restraint to the deformation. At the shoulder location, reversed moment was developed.

Figure 3. Axial thrust and Moment during the Backfill operation

Figure 4 shows axial thrust and moment distribution when backfill height reaches 3.15m, 3.75m and 4.35m. Axial thrust varies location to location as backfill height increases. For the moment, it does not vary location to location and shows symmetric with respect to the center of structure. At every construction stage maximum moment and minimum moment seems to develop at the crest and shoulder location respectively.
Figure 4. Axial thrust and Moment distribution (Backfill height: 3.15m, 3.75m, 4.35m)

Figure 5 and Figure 6 shows vehicle load test result when the soil cover reaches 0.6m and 1.0m respectively. When soil cover was 0.6m, axial thrust reaches 165kN/m at the top location and moment also reaches maximum of 30.1kN-m/m at the top location. When soil cover was 1.0m, axial thrust reaches 60kN/m at the top location and moment reaches maximum of 7kN-m/m at the top location. The result shows that only small increase of soil cover from 0.6m to 1.0 decrease axial thrust and moment dramatically and stabilized the structure. This implies that the depth of soil cover over the structure is very important for reducing the axial thrust and moment by truck live load.

Figure 5. Axial thrust and Moment during vehicle load test (Depth of soil cover : 0.6m, Truck speed: 20km/h)

Figure 6. Axial thrust and Moment vehicle load test (Depth of soil cover : 1.0m, Truck speed: 20km/h)
4 STABILITY OF STRUCTURE

Comparison of the measured result with the design calculation is shown in Table 2. In this table, plastic thrust and plastic moment capacity are also shown to check the level of developed thrust and moment during construction and vehicle load test. The measured value is maximum value and design calculation is obtained using CHBDC.

The comparison shows that the measured values are higher than the design calculation. In 0.6m soil depth, the measured axial thrust are 1.52 times higher than the design calculation and the moment are 2.5 times higher for static load. For live load case, the measured axial thrust is about 2 times higher than the calculation and the moment is 3 times higher. the measured moment from the vehicle load test is higher than the plastic capacity of the material. This implies that the structure could result in large deformation or even collapse if the depth of soil cover is shallow.

For soil depth of 1.0 case, the error between the calculation and the measurement reduces somewhat. But, still the calculation underestimates the measured value. However, as the thrust and the moment are far below the plastic capacity, the structure is considered to be stable.

From the comparison, it is concluded that the design calculation could underestimate the actual behavior and the moment from live load could lead to problem when the soil cover is shallow.

5 CONCLUSIONS

The structural behavior of semi-circular arched corrugated steel underpass during construction was obtained from the measurement of strain gages. The measured axial thrust and moment was compared with the existing design criteria. The followings are conclusions obtained from this:

1. Axial thrust and moment increase as the backfill height increase. As backfill height reaches at the top level of structure, the moment does not increase. Maximum axial thrust of 198kN/m and moment of 11.6kN-m/m occurred at the top portion of the structure.
2. Vehicle load test shows that only small increase of soil cover from 0.6m to 1.0 decrease axial thrust and moment dramatically and stabilized the structure. Adequate depth of soil cover over the structure is very important for reducing the axial thrust and moment by truck live load.
3. Comparison of the measured result with the design calculation show that the design calculation could underestimate the actual behavior and the moment from live load could lead to problem when the soil cover is shallow.

REFERENCES

CSA (2006) CHBDC(Canadian Highway Bridge Design Code), CAN/CSA-S6-06
**PAPER TITLE**  
(90 Characters Max)  
Getting Automatic Crack Detection Right for your Jurisdiction

**TRACK**  
Technology and Innovation

<table>
<thead>
<tr>
<th>AUTHOR (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>BARLOW Simon R</td>
<td>Senior Business Engineer</td>
<td>ARRB Group</td>
<td>Australia</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CO-AUTHOR(S) (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>WIX Richard P</td>
<td>Technical Specialist</td>
<td>ARRB Group</td>
<td>Australia</td>
</tr>
</tbody>
</table>

**E-MAIL**  
(for correspondence)  
Simon.barlow@arrb.com.au  
Richard.wix@arrb.com.au

**KEYWORDS:**  
- Automatic  
- Crack  
- Cracking  
- Detection  
- Reporting

**ABSTRACT:**  
Recent advancements in laser scanning technology have vastly improved our ability to collect automatic pavement cracking data at highway speeds. Although these technologies can now boast automatic crack detection (ACD) down to 1 mm width, there are still many unknowns in the transformation of this data into the type of information that will help jurisdictions make dependable maintenance decisions. ARRB has been at the forefront in adopting this ACD technology for the market. Through its Hawkeye systems, ARRB has been working with clients to produce jurisdictional appropriate information, not just raw data. Depending on its priorities, each jurisdiction will require different emphases in its outputs. In many cases, agencies will have historical practices that need to be synthesised into an automated process. ARRB has been able to work with these clients to move to using ACD successfully, while maintaining process integrity, increasing accuracy and improving efficiency. This paper presents examples of how jurisdictions have successfully used ACD data in their pavement management practices, including examples from Japan and various Australian state governments.
Getting Automatic Crack Detection Right for your Jurisdiction

Simon R Barlow and Richard P Wix
ARRB Group, Australia

1. INTRODUCTION

Automatic detection of cracks in road pavements is becoming a more commonly available technology which is able to be used by road network managers and engineers. While this has the benefit of providing increased accuracy and efficiency, a downside is that the technology is advancing faster than the roads community has been able to agree on the best way to interpret the large volumes of new data and set appropriate standards. Historical methodologies are being hastily refashioned to fit the new collection methods with widely differing results. Although the technology has proven to be internally repeatable, cross-jurisdictional comparisons are difficult due to the fundamental differences in definitions in the processing algorithms. Harmonisation of methodologies is theoretically attainable but is hampered by requirements to have continuing historically comparable data, as well as the need for the technology developers to not be impinged by restrictive standards [The World Roads Association 2013].

ARRB Group (ARRB) has been at the forefront in adopting Automatic Crack Detection (ACD) systems for the data collection market. Although it does not manufacture the sensor units themselves, ARRB has been adept at working with different jurisdictions to match their local requirements with the most appropriate equipment. Understanding the raw outputs and converting them into usable information is a critical step in making this new data accessible to decision makers.

This paper looks at four different methodologies for processing and aggregating the data obtained by the ACD. It looks at how these different methodologies affect the outputs and how different jurisdictions can interpret the data to still make justifiable management decisions. A comparison of ACD data that has been analysed in accord with the requirements of three Australian state road agencies is presented, as well as noting how the Japanese standard compares. These four standards were chosen because ARRB has significant experience with each jurisdiction, but they also are representative of the range of other standards being put forward around the world.

This paper references case studies by way of example and does not use the comparisons as a basis for recommendation of one method over another. This is because, to make a good maintenance decision on the basis of data, it is important to ensure that the processing method matches the desired outputs. In this regard, it is believed that the design process is important, as well as the ability to understand and tailor the method to suit each jurisdiction’s requirements.

2. AUTOMATIC CRACK DETECTION TECHNOLOGY

Reliable automatic crack detection data has long been unattainable to road engineers unlike other parameters such as roughness and rutting. Manual methods of collecting cracking data do not have the repeatability or robustness needed for road agencies that use performance-based specifications [Henning & Mia 2013]. In the last decade there have been advancements in sensor and computing technology which have led to the development of several viable automatic crack detection systems.

ARRB has assessed many different systems over the last decade in order to determine the applicability of each system to the wider roads industry. Systems that used digital images for either semi-automated or automated assessment were limited by technical issues such as lighting, camera resolution and environmental factors. More recently, automated systems have shown a high degree of repeatability on asphalt surfaces as well as good agreement between their respective cracking intensity results [Wix & Leschinski 2012].

One such system, the Laser Crack Measurement System (LCMS), developed by Institut National d'Optique (INO) and marketed by Pavemetrics, both of which are Canadian companies, was first introduced by ARRB in 2012 as part of its ACD offering. The ACD system consists of two high-performance 3D sensors that are fitted to the rear of the survey vehicle, 2.24 m above the pavement as shown in Figure 1.
Each LCMS consists of two main components: a high-power spread-line laser and a high-speed 3D camera mounted off-axis to the laser light source (blue). Together, the two 3D (red) laser units project a 4 m wide laser line with a 1 mm resolution across the pavement. Half of the image is captured by each camera, which interprets the distortions to the straight laser line as variations in the vertical surface profile. Measurement accuracies of 0.5 mm are possible because of the high pixel resolution.

These two sets of data are then merged to produce a 3D image. During processing, the pavement surface is divided into 5 m long by 4 m wide sections and analysed. The analysis algorithm automatically identifies any cracks and generates a crack map which can be overlaid on the pavement surface. The crack map is then processed to produce higher-level reports typically required by engineers.

The processing of ACD data is illustrated in the flow chart shown in Figure 2.

![Figure 2: ACD processing flow chart](image)
3. PROCESSING AND AGGREGATION EXAMPLES

The crack map produced by the LCMS system, much like a layer in a geographical information system (GIS), merely presents the data spatially for users to interpret to their own ends. To interpret this raw data for each jurisdiction there are a set of rules that need to be applied which form the basis of the ARRB compiling algorithm. These rules are typically set by each jurisdiction and are a mix of historical requirements, as well as some new elements that can take advantage of the new ACD technology.

When discussing ACD data the following definitions are used:

- **Cell**: smallest area in the method that is to be analysed
- **Sampled road width**: width of road that is included in the analysis (that is divided into cells)
- **Crack length**: length as reported by the LCMS sensor
- **Crack width**: the width as outputted by the LCMS sensor
- **Zone**: position of each cell transversely across the road.

The terminology used by each jurisdiction may vary from the above definitions, but the same concepts are used in each.

All four methods presented in this paper use a grid method to divide the road into cells and then process each area independently. The question of whether dealing with each area independently is important will be left for future researchers as the majority of currently used methods employ a grid method\(^1\). Each of the methods use equally spaced cells across the surveyed road width.

Table 1 presents a summary of the different parameters that each jurisdiction requires as part of their compiling algorithm. Figure 3 presents this information in a visual format. For each jurisdiction, there are additional minor adjustments that are used to filter the raw data to match required outputs.

<table>
<thead>
<tr>
<th>Jurisdiction</th>
<th>Cell length (m)</th>
<th>Cell width (m)</th>
<th>Cell area (m(^2))</th>
<th>Number of transverse cells/zones</th>
<th>Sampled road width (m)</th>
<th>Min. crack length (mm)</th>
<th>Min. crack width (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RoadCrack</td>
<td>0.50</td>
<td>0.56</td>
<td>0.28</td>
<td>4</td>
<td>2.24</td>
<td>200</td>
<td>1.0</td>
</tr>
<tr>
<td>Victoria</td>
<td>2.50</td>
<td>3.40(^2)</td>
<td>8.50</td>
<td>1</td>
<td>3.40</td>
<td>200</td>
<td>3.0</td>
</tr>
<tr>
<td>South Australia</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>3</td>
<td>3.00</td>
<td>200</td>
<td>1.0</td>
</tr>
<tr>
<td>Japan</td>
<td>0.50</td>
<td>0.50</td>
<td>0.25</td>
<td>7</td>
<td>3.50</td>
<td>50</td>
<td>n/a</td>
</tr>
</tbody>
</table>

**Table 1: Processing parameters for different jurisdictions in their compiling algorithms**

**Figure 3: Visual representation of the different jurisdiction compiling processes**

\(^1\) An example of not using independent cells can be found in the Alabama Department of Transportation (ALDOT) methodology. The ALDOT method first assesses for transverse cracks greater than 1.8 m and then looks for other cracking in the various cells second [ALDOT 2010].

\(^2\) Victoria uses the whole road width, excluding the 200 mm from the outer edge of the lane. In the comparison table, a value of 3.4 m has been substituted. This represents the width of a typical highway in Victoria.
3.1. VICTORIA
Victoria uses the coarsest method presented with each cell being 2.5 m long and covering the road width. This is done to match the historical practice of windscreens surveys, where a vehicle would drive at 20 km/h and a surveyor would visually observe cracks and record them on an analogue system. This methodology was adapted for the use of digital video cameras with post-processing in 2004 and again when they moved to using ACD in 2012. The use of the 3 mm minimum crack width is to mimic the limitations the two previous methods had in identifying fine cracks.

3.2. ROADCRACK
The RoadCrack vehicle was originally co-designed in the 1990s by the Commonwealth Scientific and Industry Research Organisation (CSIRO) and the New South Wales Roads and Traffic Authority (now Roads and Maritime Services, RMS). The system involved 4 zones of bright lighting and line scan cameras with each zone recording a nominal 500 x 600 mm square of the pavement at a 1 mm resolution. The system’s layout was restricted to the width of the survey vehicle (~2.24 m).

The RoadCrack vehicle has been used by multiple Australian jurisdictions over the years, although availability was limited as only one working RoadCrack vehicle was ever in operation at a time (the system itself has been upgraded several times during that time).

3.3. SOUTH AUSTRALIA
South Australia had used the RoadCrack vehicle on several occasions to collect network cracking data. In 2012, with ARRB’s move to ACD, it discontinued the RoadCrack methodology and designed its own methodology for interpreting the data. The positioning of four 500 x 600 mm cells across the pavement was replaced by three 1 m square cells which better captured both the truck and car wheelpaths in separate zones.

This methodology has similarities to AASHTO PP67-10, Standard Practice for Quantifying Cracks in Asphalt Pavement Surfaces from Collected Images Utilizing Automated Methods, which uses a central 1 m zone between the wheelpaths [AASHTO 2010]. AASHTO’s use of five zones has also been recommended by PIARC (The World Roads Association) in their recent paper on automatic crack detection systems [The World Roads Association 2013].

3.4. JAPAN
Japan uses a weighted percentage calculation to report as its overall cracking value, which it terms the crack ratio. The crack ratio combines the values of crocodile cracking and linear cracking, with linear cracking being weighted at only 60% of the area of the crocodile cracking. The affected area, which is a combination of these two weighted values, is divided by the total segment area to give a percentage result.

4. EFFECT OF PROCESSING METHODOLOGY ON OUTPUTS
With cracking data, there can therefore be several different answers for the same set of physical cracks. Given the collection system is calibrated and repeatable, each of these outputs could all be considered legitimate. The cracks remain the same, but the reports change.

In the example in Figure 4 and Figure 4, the same data was processed from an 18 km validation loop in Deception Bay, Queensland, Australia, using the four different methods. As can be seen, using the same data set from the ACD, the overall cracking values reported range from 29% (Victorian method) to 2% (Japanese method). If viewed by an engineer or used in pavement management software, these values would represent significantly different treatment outcomes, yet they represent the same physical cracks. Rather than accepting a particular output from the ACD as “the truth”, pavement engineers need to understand the processing methodology applied and how it affects the results.
Figure 4: Average crack percentage of Deception Bay validation loop

The Victorian method, which uses approximately an 8.5 m² cell, returns a value that is 10 times larger than the RoadCrack value which uses a 0.30 m² cell, and 4 times larger than the South Australian value which uses 1.0 m². As can be seen, the choice of cell size is therefore very important in reporting percentage cracked area.

Figure 5: Percentage breakdown of cracking widths for the Deception Bay Loop

Although the total reported percentages vary significantly, the binned breakdown of each cracking type remains the same across the various processing methods with the Victorian method slightly overemphasising the 3-5 mm wide cracks. For systems such as HDM4, which distinguish between ‘All Cracks’ and ‘Wide cracks’ (>5 mm), this could have a significant effect on the forward works program.
5. TRANSLATION TO NETWORK DECISIONS

No matter which processing method is used, jurisdictions need to be aware of the effect of each processing methodology on the reported results. Figure 6 shows a cumulative plot of the cracking results from three real networks that use the Victorian, South Australian and RoadCrack methods (Japanese data not available). Each of these datasets is very large and has been assumed to have a similar distribution of cracked segments\(^3\). As can be seen, each shows a significantly different trend when comparing percentage cracking for the worst-cracked parts of the network.

![Cumulative percentage across different networks - all cracks](image)

**Figure 6**: Cumulative percentage of cracking across different networks with different processing methodologies

The method used in Figure 6 also could be applied if a jurisdiction switches between technologies and/or methodologies and does not know how to reset the intervention criteria. If the time period between different cracking surveys is not significant (< 2 years), the cumulative plot of each collection could be graphed similar to the one shown in Figure 6 and then the new intervention criteria extrapolated from the old value, i.e. if a jurisdiction’s previous intervention criteria identified 10% of the network as being highly cracked and needing maintenance, the new intervention value could also be set based on the return of 10% of the network. Table 2 shows the variation in percentage cracking for a range of percentiles across the three different networks.

**Table 2: Table of percentile cracking values**

<table>
<thead>
<tr>
<th>Method</th>
<th>Year</th>
<th>kms</th>
<th>50th percentile (% cracked)</th>
<th>70th percentile (% cracked)</th>
<th>80th percentile (% cracked)</th>
<th>90th percentile (% cracked)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Victorian method</td>
<td>2014</td>
<td>11200</td>
<td>2.0</td>
<td>8.0</td>
<td>12.0</td>
<td>28.0</td>
</tr>
<tr>
<td>South Australian method</td>
<td>2014</td>
<td>7800</td>
<td>1.0</td>
<td>4.0</td>
<td>7.7</td>
<td>15.3</td>
</tr>
<tr>
<td>RoadCrack method</td>
<td>2014</td>
<td>614</td>
<td>0.9</td>
<td>2.5</td>
<td>4.4</td>
<td>8.8</td>
</tr>
</tbody>
</table>

The Deception Bay Loop data presented in Section 4, which is separate to the three networks, can be back-calculated to see where it would sit as a percentile in the three example networks. This data is shown in Table 3.

\(^3\) Although this assumption may be critical in understanding real network situations, for the sake of this paper these networks are being used only as theoretical examples to illustrate the effect of using different processing methodologies.
Table 3: Deception Bay loop as a percentile position in similar networks

<table>
<thead>
<tr>
<th></th>
<th>Reported average cracking (% cracked)</th>
<th>Percentile against similar network</th>
</tr>
</thead>
<tbody>
<tr>
<td>Victorian method</td>
<td>20.4(^4)</td>
<td>87</td>
</tr>
<tr>
<td>South Australian method</td>
<td>7.5</td>
<td>80</td>
</tr>
<tr>
<td>RoadCrack method</td>
<td>2.7</td>
<td>71</td>
</tr>
</tbody>
</table>

Although not perfectly aligned, the calculated percentiles are all in a similar range and indicate that the Deception Bay Loop could be considered significantly cracked when compared to the other networks. The difference in percentiles presented in this one road could be due to having significant edge cracking which would not be picked up by the RoadCrack method, which uses a 2.4 m sampled road width, but would be over-represented in the Victorian method, which has a 3.4 m sampled road width.

6. MULTI-CRITERIA INTERVENTIONS

These days, most jurisdictions use multiple-criteria intervention in order to select candidate sites. Having simple criteria, such as that presented above, would not take into account other key road parameters such as strength and surface defects. Although the Deception Bay Loop may be considered significantly cracked, this may not ultimately trigger a treatment as the road’s owner would most likely look at several additional factors before nominating it for further investigation.

The advantage of automated systems is that the compiling algorithms can produce a range of different outputs from the same data. In regards to cracking, there are a range of sub-processes to assess parameters such as crack type and crack intensity. Again, all these sub-processes would exhibit the same variability as the values presented above and road engineers would be advised to take time to understand the effect of methodologies on reported values.

7. DISCUSSION AND FURTHER RECOMMENDATIONS

Reliable and automated detection of cracking has the potential to be a major benefit to road network managers and engineers by providing increased accuracy and efficiency for their datasets. However, at this stage there are still multiple ways in which cracking data can be presented, even from the same system. In order to understand and use the data presented, engineers need to be aware of the method by which the data was processed. These processing requirements which make up the compiling algorithms should be driven by the jurisdiction’s pavement management needs and help define the medium and long term performance goals of the network.

The compiling algorithms are a key step in translating a jurisdiction’s historical practices and new requirements into usable information. Seemingly simple variations in criteria can produce widely different values, and each could be considered valid. Although direct comparisons are still hampered by historical and technological limitations, higher level statistical comparisons can be undertaken to understand the nature of the outputs relative to one another. It is important that the engineers understand that the values outputted by the system are not as strictly defined as other pavement parameters such as roughness and rutting.

The development of standards and guidelines in this area will require an element of consensus among manufacturers, providers and end users. Unfortunately, with ACD systems, consensus has not yet been reached and is proving much slower than the advancement of the technology itself. Adoption of new technologies, such as ACD, is an exciting prospect for road engineers but one that should be undertaken with a reasonable amount of diligence to understanding the system itself.

\(^4\) The Victorian method does not include cracks less than 3 mm wide and therefore the reported section average reduces from 29% (all cracks) to 20% (cracks > 3 mm width).
8. REFERENCES


Alabama Department of Transportation (ALDOT), Network-Level Pavement Condition Data Collection Procedure, ALDOT-414-04, ALDOT, Montgomery

Henning, TFP & Mia, MNU 2013, DID WE GET WHAT WE WANTED? – GETTING RID OF MANUAL CONDITION SURVEYS, Department of Civil and Environmental Engineering, The University of Auckland, Auckland


<table>
<thead>
<tr>
<th>PAPER Title (90 Characters Max)</th>
<th>Cost-Effective Safety Treatment of Foreslopes and Ditches on Low-Volume Rural Roads</th>
</tr>
</thead>
<tbody>
<tr>
<td>TRACK</td>
<td>POSITION</td>
</tr>
<tr>
<td>Karla A. LECHTENBERG</td>
<td>Research Associate Engineer</td>
</tr>
<tr>
<td>CO-AUTHOR</td>
<td>POSITION</td>
</tr>
<tr>
<td>Cody S. STOLLE</td>
<td>Post-Doctoral Research Associate</td>
</tr>
<tr>
<td>Ronald K. FALLER</td>
<td>Research Associate Professor and Director</td>
</tr>
<tr>
<td>Kevin D. SCHRUM</td>
<td>Research Engineer</td>
</tr>
<tr>
<td>E-MAIL (for correspondence)</td>
<td><a href="mailto:kpolivka2@unl.edu">kpolivka2@unl.edu</a></td>
</tr>
</tbody>
</table>

KEYWORDS: Low-Volume Roads, Safety Improvement, Benefit-to-Cost Analysis, Cost-Effective, Foreslope, and Ditch

ABSTRACT: A benefit-to-cost analysis was performed to investigate the efficacy of safety treatment alternatives for foreslopes and ditches located on roadways with traffic volumes less than 500 vehicles per day and posted speed limits of 55 mph or greater. The benefit-to-cost study was based on a parametric analysis of site characteristics found during a field survey, which included roadway geometry as well as slope/ditch geometry and frequency. Each of the 440 slope/ditch combinations was analyzed with ten traffic volumes ranging from 50 to 500 ADT. Safety treatment methods considered were: (1) "Do Nothing", which represented the baseline condition; (2) installation of W-beam guardrail; (3) installation of cable guardrail; and (4) flattening the slope.

Benefit-to-cost ratios were calculated and used to make recommendations based on the existing slope or ditch, drop height, slope/ditch length, and lateral offset of the slope/ditch away from roadway edge. Installation of a barrier was cost-beneficial for steep slopes (i.e., 1.5H:1V and 2H:1V). The "Do Nothing" option was most desirable for an unshielded ditch with a 4H:1V foreslope and either a 1H:1V or 2H:1V back slope. The road designer/engineer is encouraged to use these guidelines as a foundation for making future safety improvements but with considering site specific analysis.
COST-EFFECTIVE SAFETY TREATMENT OF FORESLOPES AND DITCHES ON LOW-VOLUME RURAL ROADS

Ms. Karla A. Lechtenberg
Dr. Cody S. Stolle
Dr. Ronald K. Faller
Dr. Kevin D. Schrum

1Midwest Roadside Safety Facility, University of Nebraska-Lincoln, Lincoln, Nebraska, USA
2University of Alabama at Birmingham, Birmingham, Alabama, USA
Email for correspondence: kpolivka2@unl.edu

1 INTRODUCTION

Roadside slopes are common features found along low-volume roads. In general, three types of slopes can be found along a roadway—foreslopes, backslopes, and transverse slopes. In addition, a foreslope may invert to a backslope within the clear zone, creating a ditch. Generally, most roadside ditches are not configured with steep slopes, although they sometimes rise into walls or steeper backslopes.

The American Association of State Highway Transportation Officials (AASHTO) Roadside Design Guide (RDG) identifies three types of foreslopes—recoverable, non-recoverable, and critical (AASHTO 2006). Recoverable foreslopes are generally 4H:1V or flatter, while non-recoverable slopes are steeper than 4H:1V but equal to or flatter than 3H:1V. Non-recoverable slopes are defined as slopes that are traversable, but a vehicle cannot easily stop or return to the roadway. Critical slopes are steeper than 3H:1V. On these slopes, the vehicle may be inclined to roll over.

The AASHTO RDG primarily provides roadside design guidance for moderate- to high-speed, high-volume highways and roadways, while providing somewhat limited guidance for low-volume, local roads and streets (AASHTO 2006). Generally-speaking, much of the latter guidance was extrapolated from higher-speed and higher-volume design guidelines. As a result, the guidelines for most rural, local roads are only loosely based on actual research results and may be impractical for local road applications due to right-of-way and financial constraints. Consequently, recommendations for implementing safety treatments have been provided for many high-speed, high-volume roadways, but safety treatments along low-volume roadways have not received the same attention due to the perception that few cost-effective treatments are available for a reasonable severity reduction.

The AASHTO Guidelines for Geometric Design of Very Low Volume Local Roads (AASHTO 2001) gives cursory coverage to roadside safety for roads with ADTs less than 400 vpd. In essence, improvements are only recommended where a documentable accident history exists. The very low traffic volumes produce sparsely populated accident histories. As a result, a single serious accident can dramatically affect the apparent need for safety treatment.

2 RESEARCH OBJECTIVE

The research objective for this study was to develop cost-effective recommendations for the safety treatment of foreslopes and ditches commonly found along roadways with traffic volumes less than 500 vpd and with posted speed limits of 55 mph (88.5 km/h) or greater by utilizing benefit-to-cost analyses. The primary research goal was to identify the most cost-effective safety treatment based on roadway geometry, foreslope and ditch geometry, and traffic characteristics.

3 FIELD INVESTIGATION

Locations

A limited field survey was undertaken to determine common characteristics of culverts and bridges placed along very low-volume roadways. The field study was conducted in Marshall County, Kansas, and Saunders and Butler counties in Nebraska. The Kansas Department of Transportation (KDOT) and Marshall County officials identified two continuous stretches of rural, roadways, which represented typical very low-volume roadways and conditions. One stretch was 8 miles (12.9 km) long, and the other segment was 13
miles (20.9 km) long. Local road officials in Butler and Saunders counties identified 55 miles (88.5 km) of very low-volume roadways in these counties. These non-paved (i.e., gravel) roadways had traffic volumes less than 500 vpd and posted speed limits of 55 mph (88.5 km/h). The field survey was limited in order to make the research study manageable, and it was not intended to be all-inclusive.

**Observations – Slope Cross-Sectional Profiles**

The sloped terrain that was observed along the low-volume roads varied in steepness, length, depth, and shape. Slope rates varied from 4H:5V to 2H:1V. Slopes flatter than 2H:1V were not recorded nor measured. Depths of the slopes ranged from 7 to 10 ft (2.1 to 3.0 m), and many slopes were often more than 100 ft (30.5 m) long. Typically, the observed slopes did not have rigid or hazardous obstacles. Examples of slopes observed during the field study are shown in Figure 1. Collected field data included: slope length, width, and height; slope steepness; lateral slope offset away from roadway edge; roadway width; shoulder width; and lane width. Additional details on the collected field data can be found in the referenced research report (Schrum et al. 2012).

![Figure 1 Typical Roadside Slopes](image)

**4 RSAP ANALYSIS**

**Overview**

This study was based on a parametric analysis of the characteristics found during a real-world site survey. Several roadway geometry and slope geometry parameters were incorporated into the Roadside Safety Analysis Program (RSAPv2) models. Once the baseline models with relevant parameters were developed, safety treatment options were identified. RSAP was used to analyze each scenario under a variety of roadway and traffic characteristics. The results of the RSAP runs were used to determine recommendations for the treatment of existing roadside slopes and ditches on very low-volume roads.

**RSAP Functional Class Coding Error**

RSAPv2 (version 2003.04.01) was used during this research study. Unfortunately, the RSAPv2 FORTRAN code contained a logical, but fixable error. When specifying a local highway, the user interface creates a model using a more severe speed and angle distribution associated with freeways. Left unmodified, this error provides results with accidents having abnormally-high severity indices. Therefore and to account for this coding error, the data file created by the user interface was modified to allow the functional class code to use the speed and angle distribution of local highways instead of freeways. A detailed explanation of this coding error as well as the associated resolution can be found in the referenced research report (Schrum et al. 2012).

**Road Geometry and Modeling**
Vertical grades and horizontal curves are common on low-volume roadways and can influence the number of accidents that occur on these roadways. Areas on hills or at crests would likely correlate with a more stringent safety treatment of roadside hazards than those found on straight, level road sections. Historical analyses of vertical curvature have shown that encroachments and crash frequency are greater on curved road sections and roads with grades as compared to straight road sections (Wright & Robertson 1976). Thus, an analysis of straight, level roads is believed to be conservative. In other words, any recommendations for treating slopes and ditches found on straight roads with level terrain should also be applied to roadways with vertical grades and horizontal curves due to a higher likelihood of serious impacts.

The roadway was modeled as a rural local road with two lanes of travel and an undivided median. It was modeled as a straight section with no vertical grade and a length of 1,500 ft (457.2 m). The roadside slope and ditch were centered in the road geometry, and the starting distances varied depending on the length of the slope parallel to the roadway. The modeled lane width was 12 ft (3.7 m), because the lane widths found during the field survey ranged from 10 ft (3.0 m) to 15 ft (4.6 m) with most 12 ft (3.7 m) wide. Shoulder width was set to 2 ft (0.6 m); since, it has been demonstrated to have little effect on the results (Sicking et al. 2009). The nominal percent of trucks was set to two percent, and the speed limit was 55 mph (89 km/h). The traffic growth factor, which is the anticipated annual traffic growth rate expressed as a percent, was zero. This means that one would not expect the amount of traffic to increase in subsequent years. The encroachment rate adjustment factor remained at the default value of 1.0, as it is intended to be used for special situations when the encroachment rate differs significantly from the average. If locations have a higher-than-average encroachment history, a value greater than 1.0 is used. Conversely, if a value of less than 1.0 is used, it means the location has a less-than-average encroachment history. However, encroachment frequencies vary widely on low-volume roadways.

**Foreslope and Ditch Geometry and Modeling**

Besides modeling the road geometry, variables necessary to develop slope and ditch models were determined. The modeling parameters for the foreslope and ditch are shown in Figures 2 and 3, respectively. The modeled values for each parameter are shown in Tables 1 and 2 for foreslopes and ditches, respectively.

![Figure 2](image-url)  
*Figure 2* Representative Slope Parameters and Locations.
Figure 3 Representative Ditch Parameters and Locations.

Table 1 Slope Modeling Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slope Profile</td>
<td></td>
<td>3H:1V, 2H:1V, 1.5H:1V</td>
</tr>
<tr>
<td>Foreslope Drop Height</td>
<td>ft (m)</td>
<td>7, 13, 20, 26 (2, 4, 6, 8)</td>
</tr>
<tr>
<td>Length</td>
<td>ft (m)</td>
<td>50, 100, 250, 500, 1000 (15, 30.5, 76, 152, 305)</td>
</tr>
<tr>
<td>Lateral Offset</td>
<td>ft (m)</td>
<td>0, 3, 7, 10 (0, 0.9, 2.1, 3)</td>
</tr>
<tr>
<td>ADT</td>
<td>vpd</td>
<td>50, 100, 150, 200, 250, 300, 350, 400, 450, 500</td>
</tr>
</tbody>
</table>

Table 2 Ditch Modeling Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ditch Profile</td>
<td></td>
<td>Foreslope – 4H:1V</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Backslope 2 – 1H:1V, 2H:1V</td>
</tr>
<tr>
<td>Backslope 1 Width</td>
<td>ft (m)</td>
<td>5, 10 (1.5, 3)</td>
</tr>
<tr>
<td>Length</td>
<td>ft (m)</td>
<td>50, 100, 250, 500, 1000 (15, 30.5, 76, 152, 305)</td>
</tr>
<tr>
<td>Lateral Offset</td>
<td>ft (m)</td>
<td>0, 3, 7, 10 (0, 0.9, 2.1, 3)</td>
</tr>
<tr>
<td>ADT</td>
<td>vpd</td>
<td>50, 100, 150, 200, 250, 300, 350, 400, 450, 500</td>
</tr>
</tbody>
</table>

*Slope Profiles*

Foreslopes were modeled using the dimensions observed in the field investigation. Five foreslope lengths and four foreslope drop heights were chosen for the analysis. The slope lengths were determined based on typical
ranges found on low-volume roads. The drop heights were defined by the available RSAP feature parameters. The transition from the travelway to the slope and from the slope back to flat roadside was modeled as a series of foreslopes. These foreslopes run perpendicular to the roadway and were chosen to replicate those found in real-world applications. The gradual decline toward the main slope was configured as 6H:1V along the roadway. Therefore, the length of the sloped transition was determined by the slope drop height. The transition slopes were then broken into sections with no more than three for the 20-ft and 26-ft (6-m and 8-m) drop heights. The lengths of these sloped transition sections were equally divided before and after the main slope. In addition, several drop heights were used to transition from level ground down to the desired drop height and were all equal in length.

*Ditch Profiles*

The best representation for ditches was to use a 4H:1V foreslope, a 4H:1V backslope, and a second backslope at 1H:1V or 2H:1V. Parallel ditches may be selected in RSAP. However, drop heights cannot be configured to model specific ditches. By using foreslopes and backslopes, the slope rate and the drop height of each component could be controlled. A 4H:1V slope was chosen due to the fact that it is fairly common for ditches on low-volume roads. This second backslope rate was varied in the study. Four widths, which included the foreslope and backslope, were determined for the ditch profile. These widths were 5, 9, 14, and 18 ft (1.5, 2.7, 4.3 and 5.5 m). The widths were based on the maximum clear zone of 18 ft for a 4H:1V slope at 55 mph (AASHTO 2006).

For a ditch width of 5 ft (1.5 m), the first backslope of 4H:1V was not used. With the given slope, the width was filled by the foreslope. For the 9-ft and 14-ft (2.7-m and 4.3-m) widths, the first backslope was 5 ft (1.5 m) wide. This backslope width determined the foreslope width and the foreslope height. For the final width of 18 ft (5.5 m), the first backslope was evaluated at widths of 5 ft (1.5 m) and 10 ft (3.0 m). The height of the second backslope was set at a constant 15 ft (4.6 m) for all configurations. The lateral offset values of 0, 3, 7, and 10 ft (0, 0.9, 2.1, and 3 m) for W-beam guardrail were determined based on the actual road width. The same lateral offsets and lengths were used for the ditches as well as the foreslopes.

*Lateral Offset*

Since a very low-volume roadway has a clear zone of 12 to 14 ft (3.7 to 4.3 m) for 6H:1V or flatter slopes based on recommendations provided in the RDG (AASHTO 2006), county and local governments are not responsible for the treatment of hazards outside of this window. The RSAP model was based on data derived from accidents on roadways with typical lane widths of 12 ft (3.7 m) or greater. Therefore, for roadways with widths greater than 24 ft (8.3 m), the road geometry was approximated by holding the lane width constant at 12 ft (4.2 m) and offsetting the slopes and ditches. The lateral offset values of 0, 3, 7, and 10 ft (0, 0.9, 2.1, and 3 m) were determined based on the actual road width. The W-beam guardrail option was considered for all four lateral offsets. However, the cable guardrail option was only considered for lateral offsets of 3, 7, and 10 ft (0.9, 2.1, and 3 m).

*Safety Treatment Options*

*Do Nothing*

Alternatives were compared to a baseline option known as the “Do Nothing” alternative. This option allowed the slopes to remain unshielded.

*Install W-Beam Guardrail*

One treatment option was the installation of a crashworthy guardrail system along the slope to prevent vehicles from traversing down onto the slope. The W-beam guardrail was modeled as a Test Level 3 (TL-3) guardrail in RSAP. A severity index (SI) multiplier of 0.7 was used. The length of the guardrail and its terminals were dependent on the slope length and width. Due to the critical slopes, guardrail lengths were selected to shield the entire intersecting slope using the guardrail runout lengths developed by Wolford and Sicking (Sicking & Wolford 1996, Wolford & Sicking 1996). The total guardrail length was the sum of the slope length and the upstream and downstream runout lengths. If the length was an odd number, it was rounded to the next increment of 12.5 ft (3.8 m) in order to use a whole number of W-beam sections. The total barrier length includes an additional 25 ft (7.6 m) due to two 12.5-ft (3.8-m) terminals.

W-Beam guardrail costs from the State Highway Agencies in Colorado, Kansas, Montana, Nebraska, Oregon, and Tennessee were averaged to obtain cost estimates for the RSAP analysis. The average cost was found to be $18.16 per linear foot ($59.58 per linear meter). A second cost for W-beam guardrail
installations of $45 per linear foot ($147.64 per linear meter) was obtained from KDOT. Since this cost was significantly higher than the other averaged states, an analysis of the same scenarios was considered using these costs.

The terminal cost was estimated at $2,100 per 37.5-ft (11.4-m) long terminal. The terminals were modeled as 12.5 ft (3.8 m); and the cost of the extra 25 ft (7.6 m) on each end was subtracted from the cost of the length of need section. Traffic control and mobilization were estimated to be 10 percent and 7.5 percent of the total cost, respectively. The traffic control cost was not to exceed $2,000. Contingency was included as 15 percent of the total cost, which covers anything that might not be covered in the other costs.

Install Cable Guardrail

The third alternative was to install cable guardrail along the slope. The cable guardrail was modeled as a TL-3 guardrail in RSAP, and the end terminals were modeled as cable guardrail terminals. The SI values for the TL-3 guardrail were updated to match the average cost of a cable median barrier crash as determined in a study of 640 cable median barrier crashes between 2002 and 2006 along Missouri roadways (Sicking et al. 2009). The average cost was given as $28,894, which resulted in an SI multiplier of 0.82 for the TL-3 guardrail.

This option was not to be used on the zero offset due to the fact that cable guardrails must be placed 4 ft (1.2 m) laterally away from the slope break point (Terpsma et al. 2008). The same method was used to determine the cable barrier length as was used for the W-beam guardrail. In this case, the end terminals for cable barriers were 16 ft (4.9 m) long. These lengths were added to the slope length to determine the total length of the barrier section, and the length of the two end terminals, totaling 32 ft (9.8 m), was added to obtain the entire barrier length.

The installation costs for the cable guardrail were broken up into three components: (1) traffic control and mobilization; (2) TL-3 low tension cable guardrail; and (3) end terminals. The costs for traffic control, mobilization, and contingency are the same as for the installation of W-beam guardrail. Costs for the cable guardrail and terminals are $22.91 per linear foot ($75.16 per linear meter) and $2,080.13 per 16-ft (4.9-m) long terminal, respectively. These costs were provided by the Missouri Department of Transportation (MoDOT).

Slope Flattening

Another treatment option was flattening of the slope. As the slope becomes flatter, the vehicle’s propensity for instability decreases, and in turn, the severity index decreases. However, the cost of slope flattening can make this alternative infeasible. Costs would be comprised of fill material, transportation of that material, labor costs, and right-of-way purchases. Each one of these components can range from almost nothing to exuberant amounts. As a result, it was difficult to conduct an explicit benefit-to-cost analysis without increasing the RSAP simulation matrix beyond a reasonable size. Instead, the engineer is referred to Roadside Grading Guidance – Phase II (Schrum et al. 2014). In the mentioned report, low-volume (less than 500 vpd) roads, guardrail had higher accident costs than even the steepest slope, thus resulting in a negative benefit-cost (B/C) ratio.

Delineation

Another treatment option could include installation of delineation devices to warn motorists of hazards located near the roadway. Based on various surveys from state departments of transportation, delineators are credited with a 30 percent and 15 percent reduction in roadside departures and run-off-road crashes on curves and straight road sections, respectively (AASHTO 2006, Labra & Mak 1980, Agent et al. 1996). As with longitudinal barriers, delineation may reduce the number of run-off-road excursions that occur on low-volume roadways as they indicate a hazard is located beyond the traveled way. Due to difficulties in quantifying the benefits of delineation, this treatment option was not considered in the RSAP analyses. Thus, an in-service performance evaluation of the delineation alternative could be used to investigate its effectiveness in a variety of low-volume roadway conditions.

5 SIMULATION RESULTS

Analyses were performed to evaluate the cost-effectiveness of various safety treatment alternatives for foreslopes and ditches found adjacent to rural, low-volume roads and within the clear zone. The analysis was conducted based on observations and site data obtained during the field study of road geometry and slope and ditch geometries. Approximately 440 slope and ditch scenarios were analyzed. Each slope and ditch
combination was analyzed with ten traffic volumes ranging from 50 to 500 ADT in increments of 50. Slope and ditch safety treatment recommendations were generated from the simulated scenarios by interpolating between the scenario results. These recommendations for slopes and ditches were made for B/C ratios of 2 and 4, as shown in Tables 3 through 7. Additional details on the simulated scenarios can be found in the referenced research report (Schrum et al. 2012).

**Foreslope Simulation Results**

For benefit-to-cost ratios of 2 and 4, the analyses indicated that there was no need to install barrier along a 3H:1V slope. For the 1.5H:1V and 2H:1V slopes, there was no need to install a barrier for roads with less than 250 ADT. Many scenarios also indicated that installing a barrier on a 2H:1V slope was unnecessary. In general, the results indicated that smaller lateral offsets and longer slopes would most likely create a scenario where a barrier was recommended for slopes of 1.5H:1V and 2H:1V.

A couple of assumptions were made during the foreslope analysis, including: (1) slopes steeper than 1.5H:1V would not be present on low-volume roadways and (2) the slope extended to a width calculated by the drop height and slope rate. Because it was assumed that the slope continued out to its greatest width based on height and rate, the length of the barrier was determined using the greatest width of the slope. This decision subsequently affects implementation costs that were given per linear feet (lf).

When the W-beam cost was analyzed as $18.16 per lf ($59.58 per linear meter), it was recommended to install W-beam guardrail instead of cable guardrail in all situations. However, when the cost of W-beam guardrail installation was analyzed as $45 per lf ($147.64 per linear meter), W-beam guardrail was only recommended for a lateral offset of less than or equal to 1.5 ft (0.5 m), where it was the only alternative. At lateral offsets of 3 ft (0.9 m) and greater, cable guardrail was also analyzed and provided lower costs. Therefore, cable guardrail was recommended over W-beam guardrail when the W-beam installation cost was $45 per lf ($147.64 per linear meter).

It is cost-effective to shield 1.5H:1V and 2H:1V slopes on roads with an ADT greater than 150 vpd. For steep slopes, B/C ratios increased linearly with ADT, and typically began around 0.25 and increasing to 4 or 5 in some cases. For a 3H:1V slope, B/C ratios were typically less than 1, and negative in many cases.

**Ditch Simulation Results**

For benefit-to-cost ratios of 2 and 4, the “Do Nothing” option was recommended for all ditch configurations. This finding held true for all foreslope widths, lengths, lateral offsets, and backslopes. Benefit-to-cost ratios were always negative, indicating that both the accident cost and the installation cost of each barrier was greater than the corresponding costs for doing nothing. The main assumption in the ditch analysis was that the ditch would be formed by 4H:1V foreslopes and backslopes, and then the backslope would continue with a steeper slope (i.e., second backslope). The 1H:1V and 2H:1V slopes that were modeled for the second backslopes had high severities, but they were offset far enough from the roadway that their severities did not significantly affect the analysis.

**6 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS**

Treatments for various foreslope and ditch configurations were considered and analyzed to determine the most cost-effective treatment for such roadside hazards. For foreslopes, treatment options included doing nothing, installing W-beam guardrail, installing a cable guardrail, and slope flattening. However for ditches, treatment options included doing nothing, installing W-beam guardrail, and installing cable guardrail.

Benefit-to-cost ratios were generated through the use of RSAPv2 and were used to determine the most cost-effective safety treatment for foreslopes and ditches in various configurations of length, depth, and lateral offset. Recommendations indicated that it was often cost-effective to leave the foreslope untreated. For the evaluated ditch configurations, it was often cost-effective to leave the ditch untreated. Further, if a foreslope or ditch is configured differently than that modeled, a site-specific benefit-to-cost analysis may be needed.

Delineation may prove to be effective for inattentive or impaired drivers by alerting motorists of an obstacle. This may reduce the number and speed of impacts due to heightened awareness. However, many crashes are the result of avoidance maneuvers, traffic violations, and mechanical failures. These crashes are typically not sensitive to delineation. Furthermore, delineation will not result in a reduction in the severity of an impact. Instead, additional investigation may be desired to evaluate how delineation may affect speed...
distribution and encroachments on very low-volume roadways where obstacle treatment guidelines indicated that removal of the existing non-crashworthy system or installation of a new crashworthy system was not a cost-effective solution. Individual analysis may be needed based on clearly-defined and quantifiable safety improvements for delineation.

Table 3  Slope Treatment Recommendations for Low-Volume, Rural Roadways, W-Beam= 18.16/If, B/C=2

<table>
<thead>
<tr>
<th>Slope Rate</th>
<th>Drop Height (ft)</th>
<th>Offset (ft)</th>
<th>Length (ft)</th>
<th>ADT (vpd)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Do Nothing</td>
</tr>
<tr>
<td>&lt; 10</td>
<td>≤ 5</td>
<td>≤ 75</td>
<td>0-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>75.1 - 175</td>
<td>0-399</td>
<td>400-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 175</td>
<td>0-299</td>
<td>300-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6 - 5</td>
<td>≤ 75</td>
<td>0-449</td>
<td>450-500</td>
</tr>
<tr>
<td></td>
<td>75.1 - 175</td>
<td>0-299</td>
<td>300-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 175</td>
<td>0-349</td>
<td>350-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.1 - 8.5</td>
<td>≤ 375</td>
<td>0-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 375</td>
<td>0-449</td>
<td>450-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 8.5</td>
<td>all</td>
<td>0-500</td>
<td></td>
</tr>
<tr>
<td>10 - 16.4</td>
<td>≤ 5</td>
<td>≤ 75</td>
<td>0-349</td>
<td>350-500</td>
</tr>
<tr>
<td></td>
<td>75.1 - 175</td>
<td>0-299</td>
<td>300-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 175</td>
<td>0-249</td>
<td>250-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.1 - 8.5</td>
<td>all</td>
<td>0-299</td>
<td>300-500</td>
</tr>
<tr>
<td></td>
<td>&gt; 8.5</td>
<td>≤ 175</td>
<td>0-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 175</td>
<td>0-399</td>
<td>400-500</td>
<td></td>
</tr>
<tr>
<td>16.5 - 22.9</td>
<td>≤ 1.5</td>
<td>≤ 75</td>
<td>0-249</td>
<td>250-500</td>
</tr>
<tr>
<td></td>
<td>&gt; 75</td>
<td>0-199</td>
<td>200-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6 - 5</td>
<td>≤ 175</td>
<td>0-249</td>
<td>250-500</td>
</tr>
<tr>
<td></td>
<td>&gt; 175</td>
<td>0-199</td>
<td>200-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.1 - 8.5</td>
<td>≤ 75</td>
<td>0-349</td>
<td>350-500</td>
</tr>
<tr>
<td></td>
<td>75.1 - 175</td>
<td>0-299</td>
<td>300-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 175</td>
<td>0-249</td>
<td>250-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 8.5</td>
<td>≤ 75</td>
<td>0-449</td>
<td>450-500</td>
</tr>
<tr>
<td></td>
<td>75.1 - 175</td>
<td>0-399</td>
<td>400-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 175</td>
<td>0-349</td>
<td>350-500</td>
<td></td>
</tr>
<tr>
<td>≥ 23</td>
<td>≤ 1.5</td>
<td>≤ 175</td>
<td>0-349</td>
<td>350-500</td>
</tr>
<tr>
<td></td>
<td>&gt; 175</td>
<td>0-199</td>
<td>200-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6 - 5</td>
<td>all</td>
<td>0-199</td>
<td>200-500</td>
</tr>
<tr>
<td></td>
<td>5.1 - 8.5</td>
<td>all</td>
<td>0-299</td>
<td>300-500</td>
</tr>
<tr>
<td></td>
<td>&gt; 8.5</td>
<td>≤ 75</td>
<td>0-399</td>
<td>400-500</td>
</tr>
<tr>
<td></td>
<td>&gt; 75</td>
<td>0-349</td>
<td>350-500</td>
<td></td>
</tr>
</tbody>
</table>
Table 3  Slope Treatment Recommendations for Low-Volume, Rural Roadways, W-Beam=$18.16/lf, B/C=2
(continued)

<table>
<thead>
<tr>
<th>Slope Rate</th>
<th>Drop Height (ft)</th>
<th>Offset (ft)</th>
<th>Length (ft)</th>
<th>ADT (vpd)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Do Nothing</td>
</tr>
<tr>
<td>&lt; 10</td>
<td>≤ 1.5</td>
<td>≤ 75</td>
<td>0-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 75</td>
<td>0-449</td>
<td>450-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 1.5</td>
<td>all</td>
<td>0-500</td>
<td></td>
</tr>
<tr>
<td>10 - 16.4</td>
<td>≤ 1.5</td>
<td>≤ 75</td>
<td>0-449</td>
<td>450-500</td>
</tr>
<tr>
<td></td>
<td>&gt; 75</td>
<td>0-299</td>
<td>300-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6 - 5</td>
<td>75.1 - 175</td>
<td>0-399</td>
<td>400-500</td>
</tr>
<tr>
<td></td>
<td>&gt; 175</td>
<td>0-299</td>
<td>350-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.1 - 8.5</td>
<td>all</td>
<td>0-449</td>
<td>450-500</td>
</tr>
<tr>
<td></td>
<td>&gt; 8.5</td>
<td>all</td>
<td>0-500</td>
<td></td>
</tr>
<tr>
<td>2H:1V</td>
<td>≤ 1.5</td>
<td>≤ 75</td>
<td>0-349</td>
<td>350-500</td>
</tr>
<tr>
<td></td>
<td>&gt; 75</td>
<td>0-299</td>
<td>300-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6 - 5</td>
<td>75.1 - 175</td>
<td>0-349</td>
<td>350-500</td>
</tr>
<tr>
<td></td>
<td>&gt; 175</td>
<td>0-299</td>
<td>300-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.1 - 8.5</td>
<td>75.1 - 175</td>
<td>0-449</td>
<td>450-500</td>
</tr>
<tr>
<td></td>
<td>&gt; 175</td>
<td>0-399</td>
<td>400-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 8.5</td>
<td>all</td>
<td>0-500</td>
<td></td>
</tr>
<tr>
<td>≥ 23</td>
<td>≤ 1.5</td>
<td>≤ 75</td>
<td>0-299</td>
<td>300-500</td>
</tr>
<tr>
<td></td>
<td>&gt; 75</td>
<td>0-299</td>
<td>250-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6 - 5</td>
<td>all</td>
<td>0-299</td>
<td>300-500</td>
</tr>
<tr>
<td></td>
<td>5.1 - 8.5</td>
<td>≤ 175</td>
<td>0-399</td>
<td>400-500</td>
</tr>
<tr>
<td></td>
<td>&gt; 175</td>
<td>0-349</td>
<td>350-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 8.5</td>
<td>≤ 175</td>
<td>0-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 175</td>
<td>0-449</td>
<td>450-500</td>
<td></td>
</tr>
<tr>
<td>3H:1V</td>
<td>all</td>
<td>all</td>
<td>0-500</td>
<td></td>
</tr>
</tbody>
</table>

Table 4  Slope Treatment Recommendations for Low-Volume, Rural Roadways, W-Beam=$18.16/lf, B/C=4

<table>
<thead>
<tr>
<th>Slope Rate</th>
<th>Drop Height (ft)</th>
<th>Offset (ft)</th>
<th>Length (ft)</th>
<th>ADT (vpd)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Do Nothing</td>
</tr>
<tr>
<td>&lt; 10</td>
<td>all</td>
<td>all</td>
<td>0-500</td>
<td></td>
</tr>
<tr>
<td>1.5H:1V</td>
<td>≤ 1.5</td>
<td>≤ 175</td>
<td>0-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 175</td>
<td>0-449</td>
<td>450-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 23</td>
<td>≤ 75</td>
<td>0-399</td>
<td>400-500</td>
</tr>
<tr>
<td></td>
<td>&gt; 75</td>
<td>0-349</td>
<td>350-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 2H:1V</td>
<td>≥ 5</td>
<td>0-500</td>
<td></td>
</tr>
</tbody>
</table>

742
<table>
<thead>
<tr>
<th>Slope Rate</th>
<th>Drop Height (ft)</th>
<th>Offset (ft)</th>
<th>Length (ft)</th>
<th>ADT (vpd)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Do Nothing</td>
</tr>
<tr>
<td>&lt; 10</td>
<td>all</td>
<td>all</td>
<td></td>
<td>0-500</td>
</tr>
<tr>
<td>10 - 16.4</td>
<td>≤ 1.5 &lt;= 175</td>
<td>0-449</td>
<td>450-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>≤ 1.5 175.1 - 375</td>
<td>0-399</td>
<td>400-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>≤ 1.5 &gt; 375</td>
<td>0-449</td>
<td>450-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6 - 5 ≤ 75</td>
<td>0-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6 - 5 75.1 - 175</td>
<td>0-449</td>
<td>450-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6 - 5 &gt; 175</td>
<td>0-299</td>
<td>300-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.1 - 8.5 ≤ 175</td>
<td>0-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.1 - 8.5 &gt; 175</td>
<td>0-449</td>
<td>450-500</td>
<td></td>
</tr>
<tr>
<td>≥ 8.5</td>
<td>all</td>
<td>0-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5:1V</td>
<td>≤ 1.5 ≤ 175</td>
<td>0-399</td>
<td>400-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>≤ 1.5 175.1 - 375</td>
<td>0-349</td>
<td>350-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>≤ 1.5 &gt; 375</td>
<td>0-399</td>
<td>400-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6 - 5 ≤ 75</td>
<td>0-399</td>
<td>400-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6 - 5 75.1 - 175</td>
<td>0-349</td>
<td>350-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6 - 5 &gt; 175</td>
<td>0-299</td>
<td>300-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.1 - 8.5 ≤ 175</td>
<td>0-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.1 - 8.5 &gt; 175</td>
<td>0-399</td>
<td>400-500</td>
<td></td>
</tr>
<tr>
<td>≥ 8.5</td>
<td>all</td>
<td>0-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td>≥ 23</td>
<td>≤ 1.5 ≤ 375</td>
<td>0-299</td>
<td>300-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>≤ 1.5 &gt; 375</td>
<td>0-349</td>
<td>350-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6 - 5 ≤ 75</td>
<td>0-299</td>
<td>300-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6 - 5 75.1 - 175</td>
<td>0-249</td>
<td>250-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6 - 5 &gt; 175</td>
<td>0-449</td>
<td>450-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.1 - 8.5 ≤ 175</td>
<td>0-399</td>
<td>400-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.1 - 8.5 &gt; 175</td>
<td>0-349</td>
<td>350-500</td>
<td></td>
</tr>
<tr>
<td>≥ 8.5</td>
<td>all</td>
<td>0-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td>≥ 23</td>
<td>≤ 1.5 ≤ 175</td>
<td>0-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≤ 1.5 &gt; 175</td>
<td>0-399</td>
<td>400-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6 - 5 ≤ 75</td>
<td>0-449</td>
<td>450-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6 - 5 75.1 - 375</td>
<td>0-399</td>
<td>400-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6 - 5 &gt; 375</td>
<td>0-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td>≥ 8.5</td>
<td>all</td>
<td>0-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2H:1V</td>
<td>≤ 1.5 all</td>
<td>0-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td>≥ 23</td>
<td>≤ 1.5 all</td>
<td>0-500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 8.5 all</td>
<td>0-500</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 6  Slope Treatment Recommendations for Low-Volume, Rural Roadways, W-Beam = $45/lf, B/C=4

<table>
<thead>
<tr>
<th>Slope Rate</th>
<th>Drop Height (ft)</th>
<th>Offset (ft)</th>
<th>Length (ft)</th>
<th>ADT (vpd)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Do Nothing</td>
</tr>
<tr>
<td>&lt; 10</td>
<td>all</td>
<td>all</td>
<td></td>
<td>0-500</td>
</tr>
<tr>
<td>10 - 16.4</td>
<td>≤ 1.5</td>
<td>≤ 175</td>
<td>0-449</td>
<td>450-500</td>
</tr>
<tr>
<td></td>
<td>175.1 - 375</td>
<td>0-399</td>
<td>400-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 375</td>
<td>0-449</td>
<td>450-500</td>
<td></td>
</tr>
<tr>
<td>1.5H:1V</td>
<td>1.6 - 5</td>
<td>≤ 75</td>
<td>0-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>175.1 - 175</td>
<td>0-449</td>
<td>450-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 175</td>
<td>0-299</td>
<td>300-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.1 - 8.5</td>
<td>≤ 175</td>
<td>0-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 175</td>
<td>0-449</td>
<td>450-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 8.5</td>
<td>all</td>
<td>0-500</td>
<td></td>
</tr>
<tr>
<td>16.5 - 22.9</td>
<td>≤ 1.5</td>
<td>≤ 175</td>
<td>0-399</td>
<td>400-500</td>
</tr>
<tr>
<td></td>
<td>175.1 - 375</td>
<td>0-349</td>
<td>350-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 375</td>
<td>0-399</td>
<td>400-500</td>
<td></td>
</tr>
<tr>
<td>≥ 23</td>
<td>1.6 - 5</td>
<td>≤ 75</td>
<td>0-399</td>
<td>400-500</td>
</tr>
<tr>
<td></td>
<td>175.1 - 175</td>
<td>0-349</td>
<td>350-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 175</td>
<td>0-299</td>
<td>300-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.1 - 8.5</td>
<td>≤ 175</td>
<td>0-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 175</td>
<td>0-399</td>
<td>400-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 8.5</td>
<td>all</td>
<td>0-500</td>
<td></td>
</tr>
<tr>
<td>≥ 23</td>
<td>≤ 1.5</td>
<td>≤ 175</td>
<td>0-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>16.4 - 22.9</td>
<td>≤ 175</td>
<td>0-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6 - 5</td>
<td>&gt; 175</td>
<td>0-399</td>
<td>400-500</td>
</tr>
<tr>
<td></td>
<td>≥ 23</td>
<td>≤ 1.5</td>
<td>0-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.6 - 5</td>
<td>≤ 75</td>
<td>0-449</td>
<td>450-500</td>
</tr>
<tr>
<td></td>
<td>&gt; 75</td>
<td>0-399</td>
<td>400-500</td>
<td></td>
</tr>
<tr>
<td></td>
<td>≥ 23</td>
<td>≤ 175</td>
<td>0-500</td>
<td></td>
</tr>
<tr>
<td>3H:1V</td>
<td>all</td>
<td>all</td>
<td>all</td>
<td>all</td>
</tr>
</tbody>
</table>

Table 7  Ditch Treatment Recommendations for Low-Volume, Rural Roadways, B/C=2 and 4

<table>
<thead>
<tr>
<th>Width (ft)</th>
<th>Backslope Rate</th>
<th>Length (ft)</th>
<th>Offset (ft)</th>
<th>ADT (vpd)</th>
</tr>
</thead>
<tbody>
<tr>
<td>all</td>
<td>all</td>
<td>all</td>
<td>all</td>
<td>all</td>
</tr>
</tbody>
</table>
ACKNOWLEDGMENTS

The authors wish to acknowledge several sources that made a contribution to this project: (1) the Midwest States Pooled Fund Program for sponsoring this research project; (2) Marshall County officials for identifying low-volume roads in Kansas; and (3) Saunders and Butler County officials for identifying low-volume roads in Nebraska.

REFERENCES


Development of Unit Weight Based Technique for Verification of Water : Cement Ratio of Fresh Concrete

**KEYWORDS:**
- unit weight
- air content
- w/c

**ABSTRACT:**
The fresh concrete water to cement ratio (w/c) determination tool is urgently needed for use in the QC/QA process at the job site. Various studies have been performed to explore the fresh concrete properties that can be used to access w/c. However, measurement of many of these properties can be complicated and time consuming. Furthermore, the extensive calibration is often needed to correlate these properties and w/c. During the course of the present study, the technique to implement the unit weight of fresh concrete as simple yet relatively accurate method to determine w/c has been developed and evaluated. The evaluation of this technique has been accomplished through the laboratory verification process. The verification of the proposed technique in the laboratory showed the capability of this technique to account for changes in the actual value of w/c caused by either the addition or the removal of water (in comparison to the design value) and the variability in absorption and moisture content of aggregates used. The verification of this technique using 58 mixes in the laboratory revealed that the minimum, maximum, absolute average, and standard error of the differences between predicted and target w/c were, respectively, 0.000, 0.042, 0.014 and 0.017. The proposed technique was verified using data collected from 89 individual concrete mixtures used during the reconstruction of the pavement on the interstate I-94 located in the northern part (near Chicago) of the state Indiana, USA. The collected data showed reasonably good relationship between the w/c (determined based on measured unit weights and air contents) and the values of flexural strength.
Development of Unit Weight Based Technique for Verification of Water-Cement Ratio of Fresh Concrete

Yohannes L. Yaphary, 1 Prof. Jan Olek, 2 Anthony Zander 3

1Sika Hong Kong Ltd., New Territories, Hong Kong
2Purdue University, West Lafayette, Indiana, USA
3Indiana Department of Transportation, Indiana, USA
Email for correspondence: yohannes@hk.sika.com

1 INTRODUCTION

The water to cement ratio (w/c) of concrete strongly influences the volume and the characteristics of capillary porosity, both of which are directly related to concrete strength and durability. Although it is a common practice to account for absorption and actual moisture content of aggregates, as well as for the amount of water added to the batch when reporting the w/c value of fresh concrete during the trial batches, this information is often not tracked during the actual production of field concrete. As a result, possibility will always exist that the actual w/c of the field mixture will be different from the design (target) value. Because of this possibility, there has always been a need for a tool (or procedure) that can verify the actual w/c value of field concrete at the time just before placement. At this time, there is no specific standardized technique for w/c determination in fresh concrete although several studies have been reported in the literature that explored various methods for obtaining this parameter (Yu et al. 2004; Melhem 1999; Philippidis & Aggelis 2003; Popovics & Popovics 1998).

The three standards test procedures that have been historically used to obtain water and/or cement content of fresh concrete (both of which are needed for w/c determination) include: ASTM C 1078 (standard test method for determining the cement content of freshly mixed concrete) (ASTM 1992a), ASTM C 1079 (standard test method for determining the water content of freshly mixed concrete) (ASTM 1992b), and AASHTO T 318 (standard test method for water content of freshly mixed concrete using microwave oven drying) (AASHTO 2002).

Since both the ASTM C 1078 and ASTM 1079 methods have been discontinued in 1998, the only standard currently available for determination of water content of fresh concrete is the AASHTO T 318 method. The procedures in ASTM C 1078 and 1079 were based on the techniques that were originally developed by Kelly and Vail in 1968 (Kelly & Vail 1968) and stipulated that the results of cement and water content for two properly conducted tests by the same operator shall not differ by more than, respectively, 2.89% and 1.50%. The AASHTO T 318 procedure is based on the study first performed by Peterson and Leftwich in 1978 (Peterson & Leftwich 1978). Because in modern ready mix plants the cement content is typically well controlled, this information can be combined with the microwave oven determined water content (after being corrected for the amount of water absorbed by the aggregates) and used to obtain the w/c. The highest accuracy in w/c determination using microwave oven technique was reported to be ± 0.01 (Nantung 1998).

The focus of the current study is on further development of the application of unit weight of fresh concrete for w/c determination purposes.

2 EXISTING W/C – UNIT WEIGHT CORRELATION AND ITS LIMITATION

In 1974 Popovics (Popovics 1974) proposed an Equation 1 that correlates w/c and unit weight of concrete.

\[ UW = 0.037 \cdot W_c \cdot \left(1 + SG_{agg} \cdot \left(\frac{16.85 \cdot (100 - a)}{W_c} - \frac{W_w}{W_c} - 0.32\right) - \frac{W_w}{W_c}\right) \]  

(1)

Where,
- \(UW\) = unit weight of the fresh concrete (lb/yd³)
- \(W_c\) = cement content (lb/yd³)
- \(W_w\) = water content (lb/yd³)
- \(SG_{agg}\) = averaged bulk specific gravity (SSD) of the aggregates
- \(a\) = air content of fresh concrete (%)

In this equation, the specific gravity of cementitious material is assumed to be 3.15. This equation was derived by keeping the weight of aggregate constant in a given and constant volume of concrete while changing the amount of
water and monitoring the resulting changes in the unit weight (Popovics 1974). Since, when the water is added to concrete in the field its volume also changes, a relationship between the unit weight and w/c that accounts for this change in volume will allow for more accurate determination of the actual w/c value. To be reasonably accurate, in addition to volume change, this relationship should also accounts for the following factors:

- The use of aggregates with absorption values that are different from those used to develop the mix design.
- The variability of moisture content of aggregates in the stockpile.

As a first step in a current study, the w/c-unit weight correlation was developed that accounts for the volume changes resulting from either addition or subtraction of water from the amount of water in the original design. During the laboratory work, the accuracy of this correlation was verified by creating groups of mixes with artificially altered values of w/c. These artificial alterations were created to represents the two previously mentioned factors that can cause changes in the field values of w/c.

3 ESTABLISHMENT OF PROCEDURE TO PREDICT W/C VALUES BASED ON THE MEASURED UNIT WEIGHT OF CONCRETE

The procedure to predict the w/c values based on the measured unit weight of concrete was developed using the following three steps:

- Establishment of unit weight-w/c relationship for basic mix.
- Adjustment of the measured unit weight.
- Combining steps 1 and 2 to predict the actual w/c value.

The process of establishing unit weight-w/c relationship was performed mathematically, using the theoretical mix composition shown in Table 1 (basic mix). This mix represents the field batch of concrete that meets the requirement of the given specification. The mix used in this study was designed to have 6.5% of total air and its w/c was 0.400.

Table 1. Basic mixture composition

<table>
<thead>
<tr>
<th>Material</th>
<th>Specific gravity</th>
<th>Weight lbs/yd³</th>
<th>Volume ft³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>3.15</td>
<td>658</td>
<td>3.36</td>
</tr>
<tr>
<td>Fine Aggregate, SSD</td>
<td>2.64 (SGₚₐ)</td>
<td>1450 (Wₚₐ)</td>
<td>8.83</td>
</tr>
<tr>
<td>Coarse Aggregate, SSD</td>
<td>2.69 (SGₚₐ)</td>
<td>1477 (Wₚₐ)</td>
<td>8.83</td>
</tr>
<tr>
<td>Water</td>
<td>1.00</td>
<td>263</td>
<td>4.23</td>
</tr>
<tr>
<td>Air</td>
<td>N/A</td>
<td>0</td>
<td>1.76</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>3848</td>
<td>27</td>
</tr>
</tbody>
</table>

The w/c-unit weight correlation was developed by changing the water amount in the basic mix design, while keeping the values of air content constant. Figure 1 shows the composite diagram of the basic batch (System #1), and the same batch altered by addition of extra water (System #2). All aggregates used in System #1 and System #2 are assumed to be in saturated surface dry (SSD) condition. The explanation of symbols used in Figure 1 is given in Table 2.

The weight of concrete ingredients per unit volume for the altered batch (system #2) can be calculated by using Equations 2 through 5. The weight of water, cementitious material, fine aggregate, and coarse aggregate (all per unit volume of concrete) are respectively labeled as Wₚₐ', Wₚₐ', Wₚₐ', and Wₚₐ'. When added, the results of these calculations will yield the unit weight of the altered batch that contains $a\%$ of total air.
Table 2. Lists of symbols used in Figure 1

<table>
<thead>
<tr>
<th>Material</th>
<th>Basic batch (before water addition)</th>
<th>Altered batch (after water addition)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Weight notation</td>
<td>Volume notation</td>
</tr>
<tr>
<td>Air</td>
<td>-</td>
<td>V_a</td>
</tr>
<tr>
<td>Water</td>
<td>W_w</td>
<td>V_w</td>
</tr>
<tr>
<td>Cementitious material</td>
<td>W_ct</td>
<td>V_ct</td>
</tr>
<tr>
<td>Fine aggregate, SSD</td>
<td>W_FA</td>
<td>V_FA</td>
</tr>
<tr>
<td>Coarse aggregate, SSD</td>
<td>W_CA</td>
<td>V_CA</td>
</tr>
</tbody>
</table>

- Amount of water added: \( \Delta W_w \)
- Total batch weight: \( W \)
- Total batch volume: \( V \)
- Air content: \( a\% \cdot V \)

\[
W'_w = \frac{W_w + \Delta W_w}{V'} = \frac{W_w + \Delta W_w}{V' \cdot (1 - a) + \frac{\Delta W_w}{\rho_w \cdot G_w}} 
\]

\[
W'_{ct} = \frac{W_{ct}}{V'} = \frac{W_{ct}}{V' \cdot (1 - a) + \frac{\Delta W_w}{\rho_w \cdot G_w}} 
\]

\[
W'_{FA} = \frac{W_{FA}}{V'} = \frac{W_{FA}}{V' \cdot (1 - a) + \frac{\Delta W_w}{\rho_w \cdot G_w}} 
\]

\[
W'_{CA} = \frac{W_{CA}}{V'} = \frac{W_{CA}}{V' \cdot (1 - a) + \frac{\Delta W_w}{\rho_w \cdot G_w}} 
\]

Figure 1. Batch component diagram before and after water addition.

System #1

System #2

\[
\begin{align*}
\text{Air} & \quad (a\% \text{ of } V) \\
\text{Water} & \quad V_a \\
\text{Cementitious material} & \quad \Delta W_w \\
\text{Fine aggregate} & \quad V_F \\
\text{Coarse aggregate} & \quad V_C
\end{align*}
\]

\[
\begin{align*}
\text{Added water} & \quad \Delta V_w \\
\text{Water} & \quad V_w \\
\text{Cementitious material} & \quad W'_w \\
\text{Fine aggregate} & \quad W'_F \\
\text{Coarse aggregate} & \quad W'_C
\end{align*}
\]
Even though Equations 2 through 5 were developed for the specific case in which water was added into the basic batch, all of these equations can be used for a case in which the amount of water is withdrawn from the basic batch by using the term ΔWw with a negative sign instead of a positive one. By using the basic mix design (from Table 1) and set of Equations 2 through 5, the compositions of five altered batches (each with different w/c values) were calculated and are shown in Table 3. The calculations shown in this table were performed assuming that all altered batches had constant air content (a = 6.5%). The compositions of the altered batches were numerically generated by adding or subtracting certain amount of water from the basic batch. This process resulted in altered batches with either lower or higher w/c values when compared to the basic batch, which had a w/c of 0.400. The unit weights of the altered batches (UW₂) were obtained by adding the weight of individual ingredients calculated by Equations 2 through 5 as shown below (Equation 6).

\[ UW_{a\%} = W_w' + W_{ct}' + W_{FA}' + W_{CA}' \]  

(6)

| Table 3. Compositions of altered batches created by changing the amount of water in the basic batch |
|-----------------------------------------------------|-----------------------------------------------------|
| **Material** | **Specific gravity** | **Amount of air (a= 6.5%)** |
| | | **Change in the amount of water (ΔW_w, lbs)** |
| | | **-13** | **-7** | **0** | **7** | **13** |
| | | **w/c of altered batch** |
| | | **0.380** | **0.389** | **0.400** | **0.411** | **0.420** |
| | **Composition volumes and unit weights of altered batches** | **Weight (lbs/yard³)** | **Weight (lbs/yard³)** | **Weight (lbs/yard³)** | **Weight (lbs/yard³)** | **Weight (lbs/yard³)** |
| Cement | 3.15 | 663 | 661 | 658 | 655 | 653 |
| Fine aggregate | 2.64 | 1462 | 1457 | 1450 | 1444 | 1438 |
| Coarse aggregate | 2.69 | 1489 | 1484 | 1477 | 1470 | 1465 |
| Water | 1 | 252 | 257 | 263 | 269 | 274 |
| Air | N/A | 0 | 0 | 0 | 0 | 0 |
| Unit weight UW_a% (lbs/yard³) | | | | | | | 3867 | 3859 | 3849 | 3838 | 3830 |

By utilizing the altered w/c and unit weights values from Table 3, the correlation between these two values was established using linear regression analysis. This correlation is expressed in mathematical form as Equation 7.

\[ W/C = -0.0010494 \cdot UW₂ + 4.439 \]  

(7)

Where,

- W/C = predicted w/c
- UW₂ = unit weight of concrete with 6.5% of air and with aggregates of the same SG as used in system #1

It should be noted that in order to apply Equation 7 to predict the w/c values of field concrete based on its measured unit weight, correction maybe needed to account for the fact that actual values of air content and specific gravities of aggregates may be different from those used in the derivation the Equation 7.

The adjustment for the differences in air content can be performed using Equation 8 shown below.

\[ UW_{a\%} = \frac{UW_{a\%'}}{(100 - a')} \cdot (100 - a) \]  

(8)

Where,

- UW_a = unit weight of concrete containing a percent of air (lbs/yard³)
- a = percent of air in the mix used to establish Equation 7 (in this case a = 6.5%)
- UW_a' = measured unit weight of concrete containing a' percent of air (lbs/yard³)
- a' = measured air content (percent)

The adjustment for the differences in specific gravities of aggregates can be performed using Equation 9.
\[ \Delta U_W = \Delta U_{W_{air}} \times \frac{(1 - a\%)\left(1 - \frac{1}{\rho_{w}}\left(\frac{1}{SG_{FA}} - \frac{1}{SG_{CA}}\right)\right) + \frac{W_{FA}}{\rho_{w}}, \left(\frac{1}{SG_{FA}} - \frac{1}{SG_{CA}}\right) + \frac{W_{CA}}{\rho_{w}}, \left(\frac{1}{SG_{FA}} - \frac{1}{SG_{CA}}\right)\right)}{(1 - a\%) + \frac{W_{FA}}{\rho_{w}}, \left(\frac{1}{SG_{FA}} - \frac{1}{SG_{CA}}\right) + \frac{W_{CA}}{\rho_{w}}, \left(\frac{1}{SG_{FA}} - \frac{1}{SG_{CA}}\right) - 1} \]  

(9)

Where,
\[ \Delta U_W \] = correction to the basic unit weight (\( U_{W_1} \)) to account for the differences in the SG of aggregates.
\[ U_{W_{air}} \] = unit weight of basic mix with a% air specified in the mix design
\[ a \] = air content that is used to develop w/c-unit weight correlation (in this case \( a = 6.5\%)\).
\[ \rho_w \] = density of water (lbs/ft\(^3\))
\[ W_{FA} \] = SSD weight fine aggregates used in the mix design (lbs/ft\(^3\))
\[ W_{CA} \] = SSD weight coarse aggregates stated in the mix design (lbs/ft\(^3\))
\[ SG_{FA} \] = SSD specific gravity of fine aggregate stated in the mix design
\[ SG_{CA} \] = SSD specific gravity of coarse aggregate stated in the mix design
\[ SG'_{CA} \] = SSD specific gravity of fine aggregate in the actual batch
\[ SG''_{CA} \] = SSD specific gravity of coarse aggregate in the actual batch

The values of unit weight calculated using Equation 8 (\( U_{W_{air}} \)) should be combined with the changes in unit weight calculated using Equation 9 (\( \Delta U_{W_{air}} \)) to yield the “corrected” value of unit weight (\( U_{W_2} \)) for use in Equation 7. This can be accomplished as shown in equation 10.

\[ U_{W_2} = U_{W_{air}} - \Delta U_{W_1} \]  

(10)

It should be noted that the use of Equations 8, 9, and 10 enables one to take a unit weight of concrete with any measured air content and produced using aggregates with any specific gravities and convert it to an equivalent (“corrected”) unit weight which is equal to the unit weight used to derive Equation 7. The details of derivations of Equation 2 to 10 can be found elsewhere (Yohannes & Olek 2012).

4 LABORATORY VERIFICATION

In order to verify the accuracy of the previously described technique of using unit weight of fresh concrete for w/c determination, four groups of laboratory mixtures were prepared as described below:

- The first group of mixes was created by adding or subtracting the predetermined amount of water from basic mix with a target w/c value of 0.400. This group of mixes was meant to represent the field concrete batch that had its designed water amount changed.
- The second group of mixes was made by assuming that the aggregates used were in SSD condition although (in reality) they were not. This approach was used to evaluate the capability of w/c-unit weight correlation that was developed by changing the amount of water in the basic mix to predict the changes in w/c of field concrete resulting from the variability of moisture content of aggregates in the stockpile.
- The third group of mixes duplicated mixes from the second group but was made by changing the type of coarse aggregate to the one with different values of specific gravity and absorption than those used to develop w/c-unit weight correlation. Two types of coarse aggregates with different values of specific gravity and absorption, compared to those stated for the basic mix design (Table 1), were used. These types of coarse aggregates were steel slag (with SG’ CA of 3.57 and absorption of 1.7%) and limestone (with SG’ CA of 2.72 and absorption of 1.0%). The tests on the mixes in this group were performed to determine the capability of the developed w/c-unit weight correlation to predict the alteration of w/c caused by using the aggregates with absorption values different from those used for the basic mix design. Furthermore, the tests on the mixes in this group were also used to evaluate the applicability of Equation 10.
- The fourth group of mixes was created by combining variables used in the first and second or those used in first and third groups.

During the verification part, a total of 58 different mixtures were prepared and their w/c were predicted. Table 4 presents the predicted and target w/c values and the values of \( \Delta w/c \) (the difference between predicted and target w/c) for these 58 mixes. The batches with codes A and CS (A1 to A9 and CS1 to CS5), B and F (B1 to B11 and F1 to F6), and D and R (D1 to D5, R1A to R1C, and R2A to R2C) represent, respectively, the first, second, and third group of mixes. The other batches were the fourth group of mixes. The batches with codes D and E (D1 to D5 and E1 to E6) were produced using limestone coarse aggregate and the batches with codes F, G, and R (F1 to F6, G1 to G4, R1A to R1C, and R2A to R2C) were produced using steel slag coarse aggregate. The other batches were prepared using the same coarse aggregate as the one used to prepare the basic mix (i.e. the dolomite coarse aggregate).

<p>| Table 4. Predicted and target values of w/c and ( \Delta w/c ) |</p>
<table>
<thead>
<tr>
<th>Mix code (group) - SG&lt;sub&gt;CA&lt;/sub&gt;</th>
<th>Predicted w/c</th>
<th>Target w/c</th>
<th>Δ w/c†</th>
<th>Mix code (group) - SG&lt;sub&gt;CA&lt;/sub&gt;</th>
<th>Predicted w/c</th>
<th>Target w/c</th>
<th>Δ w/c†</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1(1) - 2.69</td>
<td>0.398</td>
<td>0.380</td>
<td>+0.017</td>
<td>D4(3) - 2.72</td>
<td>0.420</td>
<td>0.432</td>
<td>+0.012</td>
</tr>
<tr>
<td>A2(1) - 2.69</td>
<td>0.394</td>
<td>0.390</td>
<td>+0.004</td>
<td>D5(3) - 2.72</td>
<td>0.416</td>
<td>0.432</td>
<td>-0.015</td>
</tr>
<tr>
<td>A3(1) - 2.69</td>
<td>0.408</td>
<td>0.400</td>
<td>+0.008</td>
<td>E1(4) - 2.72</td>
<td>0.416</td>
<td>0.404</td>
<td>+0.013</td>
</tr>
<tr>
<td>A4(1) - 2.69</td>
<td>0.429</td>
<td>0.410</td>
<td>+0.019</td>
<td>E2(4) - 2.72</td>
<td>0.423</td>
<td>0.417</td>
<td>+0.006</td>
</tr>
<tr>
<td>A5(1) - 2.69</td>
<td>0.380</td>
<td>0.380</td>
<td>+0.000</td>
<td>E3(4) - 2.72</td>
<td>0.420</td>
<td>0.431</td>
<td>-0.011</td>
</tr>
<tr>
<td>A6(1) - 2.69</td>
<td>0.383</td>
<td>0.380</td>
<td>+0.003</td>
<td>E4(4) - 2.72</td>
<td>0.441</td>
<td>0.458</td>
<td>-0.018</td>
</tr>
<tr>
<td>A7(1) - 2.69</td>
<td>0.398</td>
<td>0.400</td>
<td>-0.002</td>
<td>E5(4) - 2.72</td>
<td>0.423</td>
<td>0.420</td>
<td>+0.003</td>
</tr>
<tr>
<td>A8(1) - 2.69</td>
<td>0.405</td>
<td>0.400</td>
<td>+0.005</td>
<td>E6(4) - 2.72</td>
<td>0.434</td>
<td>0.434</td>
<td>+0.000</td>
</tr>
<tr>
<td>A9(1) - 2.69</td>
<td>0.440</td>
<td>0.420</td>
<td>+0.020</td>
<td>F1(2) - 3.57</td>
<td>0.435</td>
<td>0.430</td>
<td>+0.006</td>
</tr>
<tr>
<td>B1(2) - 2.69</td>
<td>0.440</td>
<td>0.458</td>
<td>-0.018</td>
<td>F2(2) - 3.57</td>
<td>0.389</td>
<td>0.430</td>
<td>-0.041</td>
</tr>
<tr>
<td>B2(2) - 2.69</td>
<td>0.460</td>
<td>0.458</td>
<td>+0.003</td>
<td>F3(2) - 3.57</td>
<td>0.423</td>
<td>0.430</td>
<td>-0.007</td>
</tr>
<tr>
<td>B3(2) - 2.69</td>
<td>0.470</td>
<td>0.507</td>
<td>-0.036</td>
<td>F4(2) - 3.57</td>
<td>0.418</td>
<td>0.430</td>
<td>-0.012</td>
</tr>
<tr>
<td>B4(2) - 2.69</td>
<td>0.429</td>
<td>0.450</td>
<td>-0.020</td>
<td>F5(2) - 3.57</td>
<td>0.381</td>
<td>0.404</td>
<td>-0.023</td>
</tr>
<tr>
<td>B5(2) - 2.69</td>
<td>0.443</td>
<td>0.456</td>
<td>-0.013</td>
<td>F6(2) - 3.57</td>
<td>0.381</td>
<td>0.404</td>
<td>-0.023</td>
</tr>
<tr>
<td>B6(2) - 2.69</td>
<td>0.443</td>
<td>0.456</td>
<td>-0.013</td>
<td>G1(4) - 3.57</td>
<td>0.372</td>
<td>0.414</td>
<td>-0.042</td>
</tr>
<tr>
<td>B7(2) - 2.69</td>
<td>0.446</td>
<td>0.456</td>
<td>-0.009</td>
<td>G2(4) - 3.57</td>
<td>0.398</td>
<td>0.414</td>
<td>-0.017</td>
</tr>
<tr>
<td>B8(2) - 2.69</td>
<td>0.457</td>
<td>0.456</td>
<td>+0.001</td>
<td>G3(4) - 3.57</td>
<td>0.285</td>
<td>0.278</td>
<td>+0.008</td>
</tr>
<tr>
<td>B9(2) - 2.69</td>
<td>0.415</td>
<td>0.395</td>
<td>+0.021</td>
<td>G4(4) - 3.57</td>
<td>0.268</td>
<td>0.278</td>
<td>-0.009</td>
</tr>
<tr>
<td>B10(2) - 2.69</td>
<td>0.409</td>
<td>0.395</td>
<td>+0.015</td>
<td>C1(1) - 2.69</td>
<td>0.399</td>
<td>0.400</td>
<td>-0.001</td>
</tr>
<tr>
<td>B11(2) - 2.69</td>
<td>0.413</td>
<td>0.395</td>
<td>+0.018</td>
<td>C2(1) - 2.69</td>
<td>0.468</td>
<td>0.450</td>
<td>+0.018</td>
</tr>
<tr>
<td>C1(4) - 2.69</td>
<td>0.355</td>
<td>0.338</td>
<td>+0.017</td>
<td>C3(1) - 2.69</td>
<td>0.518</td>
<td>0.500</td>
<td>+0.018</td>
</tr>
<tr>
<td>C2(4) - 2.69</td>
<td>0.457</td>
<td>0.472</td>
<td>-0.015</td>
<td>C4(1) - 2.69</td>
<td>0.565</td>
<td>0.550</td>
<td>+0.015</td>
</tr>
<tr>
<td>C3(4) - 2.69</td>
<td>0.422</td>
<td>0.403</td>
<td>+0.020</td>
<td>C5(1) - 2.69</td>
<td>0.583</td>
<td>0.600</td>
<td>-0.017</td>
</tr>
<tr>
<td>C4(4) - 2.69</td>
<td>0.443</td>
<td>0.416</td>
<td>+0.027</td>
<td>R1A(3) - 3.57</td>
<td>0.385</td>
<td>0.410</td>
<td>-0.025</td>
</tr>
<tr>
<td>C5(4) - 2.69</td>
<td>0.453</td>
<td>0.430</td>
<td>+0.023</td>
<td>R1B(3) - 3.57</td>
<td>0.402</td>
<td>0.410</td>
<td>-0.008</td>
</tr>
<tr>
<td>C6(4) - 2.69</td>
<td>0.457</td>
<td>0.444</td>
<td>+0.013</td>
<td>R1C(3) - 3.57</td>
<td>0.407</td>
<td>0.410</td>
<td>-0.003</td>
</tr>
<tr>
<td>D1(3) - 2.72</td>
<td>0.423</td>
<td>0.446</td>
<td>-0.022</td>
<td>R2A(3) - 3.57</td>
<td>0.391</td>
<td>0.410</td>
<td>-0.019</td>
</tr>
<tr>
<td>D2(3) - 2.72</td>
<td>0.423</td>
<td>0.446</td>
<td>-0.022</td>
<td>R2B(3) - 3.57</td>
<td>0.391</td>
<td>0.410</td>
<td>-0.019</td>
</tr>
<tr>
<td>D3(3) - 2.72</td>
<td>0.420</td>
<td>0.432</td>
<td>-0.012</td>
<td>R2C(3) - 3.57</td>
<td>0.412</td>
<td>0.410</td>
<td>+0.002</td>
</tr>
</tbody>
</table>

†Δ w/c is the difference between predicted and target w/c.

The unit weight and air content of the mixes made during the laboratory study were measured using the following procedures (see Figure 2):

- The unit weight container (±0.25 fl3) was filled with water, covered with glass and the weight of the entire assembly was recorded as <i>W</i><sub>1</sub>.
- The container was emptied and filled with concrete up to approximately 80% of its volume; the weight of added concrete was then recorded as <i>W</i><sub>sample</sub>.
- Water was added to the concrete until container was about 90% full.
- The concrete/water mixture was then stirred to force the air to rise to the surface.
- In order to eliminate the foam produced as a result of the stirring process, the surface of the concrete-water mix was sprayed with an anti-foaming agent. In this study, isopropyl alcohol was used.
- The sample was stirred again in order to make sure that all of the air had been removed. The spraying process was repeated again (if necessary).
- More water was added until the container was completely full.
- The full container was weighed and its weight was recorded as <i>W</i><sub>2</sub>.
Note:

\[ 
W_1 = W_{\text{cont}} + W_{\text{watercont}} + W_p \\
W_{\text{cont}} = \text{weight of unit weight container} \\
W_{\text{watercont}} = \text{weight of water needed to fully fill up the unit weight container} \\
W_p = \text{weight of glass plate} \\
W_2 = W_{\text{cont}} + W_{\text{sample}} + W_{\text{wadd}} + W_p \\
W_{\text{sample}} = \text{weight of concrete sample} \\
W_{\text{wadd}} = \text{weight of water added to fully fill up the unoccupied space of unit weight by the concrete sample} \\
\]

Figure 2. Illustrations of the Procedure to Determine the Values of \( W_1 \), \( W_2 \), and \( W_{\text{sample}} \)

After the values of \( W_{\text{sample}} \), \( W_1 \), and \( W_2 \) were obtained, the unit weight and air content were calculated using Equations 11 and 12, respectively. The \( W_{\text{watercont}} \) is the weight of water needed to fully fill the unit weight container. The term air content \( (a') \) represents the percentage of the volume of unit weight container not occupied by concrete.

\[
UW_{a'} = \frac{W_{\text{sample}} \cdot \rho_w}{W_{\text{watercont}}} \cdot 100\
\]

\[(11)\]

\[
a' = \frac{W_{\text{sample}} - (W_1 - (W_2 - W_{\text{sample}}))}{W_{\text{watercont}}} \cdot 100\%
\]

\[(12)\]

In this study the procedures illustrated in Figure 2 and described by Equations 11 and 12 were used to obtain the values of \( UW_{a'} \) and \( a' \), because this technique does not require the knowledge of the aggregates correction factor (ACF). The knowledge of the ACF is required when using the pressure method as described in AASHTO T 152 (standard method of test for air content of freshly mixed concrete by the pressure method) (AASHTO 2005b). If the aggregates correction factor is precisely known, the techniques to measure unit weight and air content as respectively described in AASHTO T 121 (standard method of test for density (unit weight), yield, and air content (gravimetric) of concrete) (AASHTO 2005a) and AASHTO T 152 can be used to obtain the values of \( UW_{a'} \) and \( a' \). In this case, the \( UW_{a'} \) is the unit weight measured and \( a' \) is the measured air content after applying the aggregates correction factor.

The example of calculations used to predict the w/c of batch E1 using the unit weight and air content that were measured following the above proposed method is provided below. The data needed for these calculations were as follows:

\[
W_{\text{watercont}} = 15.54 \text{ lbs} \\
W_{\text{sample}} = 28 \text{ lbs} \\
W_1 = 7.65 \text{ lbs} + 15.54 \text{ lbs} + 2.71 \text{ lbs} = 25.9 \text{ lbs} \\
W_2 = 42.47 \text{ lbs}, \\
\rho_w = 62.27 \text{ lbs/ft}^3 \\
\]

By using Equation 11, the unit weight for specimen E1 can be calculated:

\[
UW_{a'} = \frac{28 \cdot 62.27}{15.54} = \frac{1122.0}{\text{ft}^3} = \frac{3029}{\text{yd}^3} \\
\]
By using Equation 12 and all of the data provided for specimen E1, the air content for specimen E1 can be calculated as:

$$a'\% = \frac{15.54 - (25.9 - (42.47 - 28))}{15.54} \cdot 100\% = 26.5\%$$

The unit weight of specimen E1 with 6.5% air content can be calculated using Equation 8 as shown below.

$$UW_{a'\%} = \frac{3029.35}{(1 - 6.5\%)} \cdot (1 - 6.5\%) = 3851 \text{ lbs/yd}^3$$

In order to predict the w/c of batch E1 using Equation 7, the unit weight of batch E1 with 6.5% air content ($UW_{a'\%}$) needs to be corrected by $\Delta UW_1$ (Equation 9). The result is shown below. The value of $W_{FA}$ and $W_{CA}$ are those stated for the basic mix design in Table 1. This correction needs to be done because $SG'_{FA}$ and $SG'_{CA}$ are not equal to $SG_{FA}$ and $SG_{CA}$.

$$\Delta UW_1 = 3851 \cdot \left[ (1 - 6.5\%) + \frac{1450}{62.27 \cdot 27} \cdot \left( \frac{1}{2.64} - \frac{1}{2.64} \right) + \frac{1477}{62.27 \cdot 27} \cdot \left( \frac{1}{2.72} - \frac{1}{2.69} \right) - 1 \right]$$

$$\Delta UW_1 = 18 \text{ lbs/yd}^3$$

The corrected unit weight of specimen E1 with 6.5% air content and equivalent specific gravities of fine and coarse aggregates equal respectively, to 2.64 ($SG_{FA}$) and 2.69 ($SG_{CA}$) (the specific gravities of aggregates used to develop w/c-unit weight as it is formulated in Equation 7), is calculated by subtracting the unit weight of 3851 lbs/yd$^3$. The result is 3833 lbs/yd$^3$. Once the unit weight of sample E1 has been corrected, its w/c can be predicted by using Equation 7.

$$\frac{W}{C} = -0.0010494 \cdot 3833 + 4.439$$

$$\frac{W}{C} = 0.416$$

5 FIELD VERIFICATION

The w/c of the concretes used for project I-94 were verified by using their measured unit weight and air content data. Total of 89 mixtures that have concrete mix design (CMD) as shown in Table 5 were included in this verification. The methods used to measure the unit weights and air contents in this projects followed AASHTO T 121 and AASHTO T 152, respectively. In addition to the data of unit weights and air contents, flexural strength data were also obtained for these 89 mixtures. The plots between w/c (determined based on measured unit weights and air contents) and measured flexural strengths show reasonable tendency as shown in Figure 3.

<table>
<thead>
<tr>
<th>Material</th>
<th>Specific gravity</th>
<th>$W_{FA}$ (lbs/yd$^3$)</th>
<th>$W_{CA}$ (lbs/yd$^3$)</th>
<th>$W_{FA}$ (ft$^3$)</th>
<th>$W_{CA}$ (ft$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>3.15</td>
<td>440</td>
<td>2.24</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fly Ash</td>
<td>1.91</td>
<td>71</td>
<td>0.60</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fine Aggregate, SSD</td>
<td>2.56 ($SG_{FA}$)</td>
<td>1345 ($W_{FA}$)</td>
<td>8.44</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Coarse Aggregate, SSD</td>
<td>2.76 ($SG_{CA}$)</td>
<td>1849 $W_{CA}$</td>
<td>10.76</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Water</td>
<td>1.00</td>
<td>204</td>
<td>3.28</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Air</td>
<td>N/A</td>
<td>0</td>
<td>1.76</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>3909</td>
<td>27</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
6 CONCLUSIONS

During the course of the current study, the technique to implement the unit weight of fresh concrete for w/c determination has been developed and verified. The evaluations of this technique have been performed through the laboratory verifications. The verification of the proposed technique using 58 laboratory mixtures showed that the proposed technique is capable to predict the changes in w/c of fresh concrete due to variation in the total water content, specific gravity and adsorption of aggregates used. The collected results revealed that the minimum, maximum, absolute average and standard error of the differences between predicted and target w/c were, respectively, 0.000, 0.042, 0.014 and 0.017. The standard error was calculated using Equation 13 (Dowell & Cramer 2002).

\[
SE = \left( \frac{\sum \left( \frac{\text{measured} \ w}{c} - \frac{\text{target} \ w}{c} \right)^2}{n} \right)^{\frac{1}{2}} \tag{13}
\]

Where,
\[
\begin{align*}
\text{SE} &= \text{standard error} \\
\text{n} &= \text{number of tests}
\end{align*}
\]

The advantages of implementing unit weight to determine w/c are:
- The method is field-worthy because it is very robust and does not require sophisticated equipment.
- The method is fast (requires less than 15 minutes), simple to perform, and inexpensive.
- Independent test is not required if the procedures to measure the unit weight and air content as, respectively, methods, described in AASHTO T 121 M and T152 are used and if there are a certainty in specific gravities and correction factor (ACF) values of used aggregates are known.

The disadvantage of this technique is the sensitivity of unit weight to the values of specific gravities of aggregates used, which need to be verified prior to predicting w/c. This SG verification process can be time consuming and thus negatively impacting the implementation of unit weight as a technique for w/c determination.

7 ACKNOWLEDGEMENTS

This work was supported by the Joint Transportation Research Program administered by the Indiana Department of Transportation and Purdue University. The content of this paper reflects the views of the authors, who are responsible for the facts and the accuracy of the data presented herein, and does not necessarily reflect the official views or policies of the Federal Highway Administration and the Indiana Department of transportation, nor do the content constitutes a standard, specification, or regulation.
REFERENCES

AASHTO Standard Designation T 121M/T 121-05. Standard Method of Test for Density (Unit Weight), Yield, and Air Content (gravimetric) of Concrete. The American Association of State Highway and Transportation Officials, 2005.


Popovics, S. Proportioning Concrete for a Specified Cement Content and/or a Specified Unit Weight. Proportioning Concrete Mixes, ACI SP-46-4, January 1, 1974, pp. 47-63.


Road Traffic Accidents and Fatality Rates in Libya
Ahmed Mohamed Gadi¹, Ahmed Ali Bensaied¹, Rouida Ahmed Gadi²
¹Professor, ²Graduate Student, Civil Eng. Dept. Tripoli University, Tripoli, Libya
Email: am_gadi55@yahoo.com

Abstract: In the last quarter of the twentieth century, most of the Libyan cities and villages have been connected by road network. These roads were the major drive force for economical and social development all over the country. Unfortunately, despite of this positive role, these roads are the main cause of death and loss of about 2% of Libyan national annual income.

This is because most of these roads, if not all, are lacking the basic elements of safety needs, besides; maintenance programs were not implemented. In addition, and probably the most important factors, the driver’s behaviors and driving skills and habit are not at standard up to the level required for safety driving. These mentioned conditions affect the passenger's safety, and led to catastrophic accidents.

The main objective of this paper is to demonstrate the latest Libyan accidents statistics and the losses of life as well as compares the Libyan fatality rate with that of some Asian, African and European countries. Data for this purpose were collected from Libyan traffic police department and the international traffic safety data analysis group.

The results of these comparisons show that the Libyan fatality rate is the highest among the compared countries. Several countermeasures / recommendations on how to improve traffic safety on Libyan roads are also discussed in this paper.

Key words: Traffic Safety, Accidents, Fatality Rates.

1. Status of Road Accidents in Libya
Traffic accident in Libya has been increasing at the average rate of 5.0% each year during the last 10 years as listed in Table 1. The total number of road accidents had increased from 11643 accidents in 2004 to 15665 accidents in 2010 that is about 134.5% increase of accidents over those 7 years. Then this number had dropped down to 3154 during the year 2011 due to the Libyan revolution (war) and the consequences of shortage of fuel and almost no traffic movement between cities. Once things started to be stabilized and motor movement started, the number of accidents rose again to be 5458 at 2012 and then doubled up to 10105 by the end of the year 2013. The number of fatalities also had increased from 1785 in 2004 to 3606 in 2013, to reach about 10 persons each day during the last year 2013.

Table 1: Libyan road accident statistics

<table>
<thead>
<tr>
<th>Year</th>
<th>Total number of traffic accident</th>
<th>Number of Deaths</th>
<th>No. of Fatalities per 100,000 Populations</th>
</tr>
</thead>
<tbody>
<tr>
<td>2004</td>
<td>11643</td>
<td>1785</td>
<td>34.9</td>
</tr>
<tr>
<td>2005</td>
<td>11898</td>
<td>1800</td>
<td>34.5</td>
</tr>
<tr>
<td>2006</td>
<td>11982</td>
<td>1866</td>
<td>35.1</td>
</tr>
<tr>
<td>2007</td>
<td>13556</td>
<td>2138</td>
<td>39.4</td>
</tr>
<tr>
<td>2008</td>
<td>13352</td>
<td>2332</td>
<td>42.4</td>
</tr>
<tr>
<td>2009</td>
<td>13664</td>
<td>2301</td>
<td>41.1</td>
</tr>
<tr>
<td>2010</td>
<td>15665</td>
<td>2499</td>
<td>43.9</td>
</tr>
<tr>
<td>1122</td>
<td>4213</td>
<td>2211</td>
<td>20.1</td>
</tr>
<tr>
<td>1121</td>
<td>1311</td>
<td>4111</td>
<td>50.3</td>
</tr>
<tr>
<td>1124</td>
<td>21211</td>
<td>4313</td>
<td>60.1</td>
</tr>
</tbody>
</table>

Source: Libyan Traffic & Licensing Police Department (February, 2014).

The increase of road accidents and fatality were due to the population growth (from 3.8 in 1990 to 5.9 million in 2013) which results a development of housing and industrial sectors, accordingly, created a need for travel movement interaction. The total number of registered vehicles had drastically increased from 161,575 in 1990 to 2,141,313 vehicles in 1124. The Comparison of the death rates per 100,000 population fatalities for the years 1113 and 1123 shows a drastic increase from 34.9 to 60.1. According to the Libyan
Traffic & Licensing Police Department 2014 report, the total number of killed persons during the years 1990 to 2013 is 41641 which represents about 0.7% of total Libyan population.

A corrective and/or preventive programs to mitigate and reduce accidents, is urgently needed, and is completely depends on our knowledge of the accidents causes from the collected data. Accordingly, the traffic accident information should be gathered, reported, filed and stored in a data bank, so it will be available for the specialists and researchers to use in a future work concerning any strategic plans for traffic safety and traffic accident reduction.

2. Accident data collection

The procedure of accident data collection in Libya is a responsibility of the technical traffic police department. Usually the information filed either at accident spot or at later time, when the involved parties report the occasion at the police station.

In Libya, usually the recorded accident information are; the names of drivers involved in the accident, their age, the number of vehicle occupants, time of accident, speed, vehicle plate license, the mandatory insurance card number and some brief description information of the circumstances of the accident such as weather, date…etc.

Although, all the mentioned information is usually registered and stored, nevertheless is not in a correct way, not in a proper order to use by researchers and not easy to access, accordingly it needs some good attention and needs to be sorted out and stored, in electronic devices, to be accessible for those concerned with this subject.

3. Road safety aspects in Libya

Libya is a huge country in terms of area compared to any of European countries. It occupies about 1,750,000 square kilometer of land. All of the goods movement and most of the passenger travel are made by roads. The road net work consists of about 30,000 km of paved roads, connecting all the cities and most of the remote villages, serving a population of 5.9 million. Most of these roads were designed and constructed, during nineteen seventies where (In Libya) the safety measures were not very much considered.

The lack of the basic safety elements, led to catastrophic accidents caused lots of deaths and a loss of wealth. Examples of these elements are, marking lines, urban street lighting, traffic control devices, school signs and pedestrian crossing. Furthermore, the existence of dangerous curves and hazardous locations without alerting signs, in addition to the lack of a periodical preventive and corrective maintenance of the road pavement and shoulders, all these elements, either each alone or together, are the main cause of accidents.

During the year 2013 the number of fatal accidents was 2762 representing a rate of 27.3% of the total accidents (10105) in Libya. The total number of road casualties was 6178 case which represents 61.2% of the total accidents. While the Property Damage Only, PDO, was 1165 representing 11.5% of the total accident, as shown in Table 2. By comparing the year 2013 accidents related figures with those of the year 2012, as presented in Table 2, we find out that there is a noticeable increasing number of total accidents 4647 and an increasing of 412 death accident over the year 2012. These accidents caused a total number of 3606 persons died in the year 2013. This number is 586 higher than the total number of fatality of the previous year 2012.

This is very dangers indication, and if some measures were not taken, by laying out a good and measurable strategy, to reduce this climbing rate, the traffic accident in Libya, with high confident, will be in future, the first cause of death at this country

Table 2: Numbers and percentages of accidents for the years 2012&2013.

<table>
<thead>
<tr>
<th>Year</th>
<th>PDO</th>
<th>Serious Injury</th>
<th>Fatal accidents</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>2012</td>
<td>2432</td>
<td>2343</td>
<td>2350</td>
<td>5458</td>
</tr>
<tr>
<td>%</td>
<td>25.1%</td>
<td>31.8%</td>
<td>43.1%</td>
<td>100%</td>
</tr>
<tr>
<td>2013</td>
<td>1165</td>
<td>6178</td>
<td>2762</td>
<td>21211</td>
</tr>
<tr>
<td>%</td>
<td>11.5%</td>
<td>61.2%</td>
<td>27.3%</td>
<td>100%</td>
</tr>
</tbody>
</table>

Source: Libyan Traffic & Licensing Police Department (February, 2014).
4. Comparing Libyan and Other Countries Fatalities

Traffic accidents have remained one of the leading contributing factors of deaths in Libya. These accidents are accounting for more than 99.9% of fatalities and almost 100% of serious injuries in transport-related occurrences according to Libyan traffic police department. During the last 10 years, road transportation fatalities and injuries have been increased by 32% per unit of population basis. Comparing the Libyan and Sweden fatality rates per 100,000 populations, for the year 2013 (almost both countries having the same population) it is clear that the Libyan fatalities 60.1 is almost 12 times the fatality rates of Sweden roads 5, according to the unit population basis.

Furthermore, comparing Libyan rates with adjacent country Egypt, it is clear that, the Libyan rate of road fatality is more than 3 times higher than that of Egyptian rate. Knowing that, the Sweden population 9 million is not far different from Libyan population and that of Egypt is 85 million very far of the Libyan population about 6 million. North of the Mediterranean Sea, and their fatality rates, such as Italy, France and Spain 8.3, 5.6, 9 respectively, all look very low when comparing with Libyan rate.

Again, when taking the population of these mentioned countries into consideration, it is very obvious that they are larger than Libya. Figure 1 demonstrates the fatality rates per 100,000 populations for some African, Asian, European and North American countries.

![Figure 1 Number of deaths per 100,000 populations](image)

Looking to Table 3 and Figure 2 up to figure 7 which represent the number of road killing over the span of the last ten years, as an example, for the countries of Oman, Denmark, Spain, Australia, France and UK against Libya. It can be noticed clearly the difference of a good, effective, and successful, national road safety plan that implemented at those countries and in the other hand set doing nothing (Libyan plan) for the last 40 years.

All figures show the decline in road collision deaths for those mentioned countries data and, as it looks, the reduction in road fatality is expected to continue, according to these trends. While, in the same time, the Libyan road killing data trend show the opposite direction, rising drastically, and is expected to continue, unless traffic safety authorities in Libya started to be committed to improve road safety, introducing important measures, follow a clear measurable safety strategy and seek the help from other developed countries, such as any of the European union (Mediterranean) countries.
Table (3) Population & number of road traffic accident fatalities per year

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Libya</td>
<td>51.6</td>
<td>.411</td>
<td>.476</td>
<td>.711</td>
<td>.755</td>
<td>8.17</td>
<td>8118</td>
<td>811.</td>
<td>8122</td>
<td>8451</td>
<td>1181</td>
</tr>
<tr>
<td>Oman</td>
<td>111.</td>
<td>781</td>
<td>261</td>
<td>26.</td>
<td>427</td>
<td>57.</td>
<td>572</td>
<td>514</td>
<td>647</td>
<td>671</td>
<td>122</td>
</tr>
<tr>
<td>Denmark</td>
<td>6162</td>
<td>118</td>
<td>152</td>
<td>11.</td>
<td>115</td>
<td>115</td>
<td>115</td>
<td>111</td>
<td>866</td>
<td>881</td>
<td>.54</td>
</tr>
<tr>
<td>Spain</td>
<td>1418</td>
<td>6111</td>
<td>1412</td>
<td>1118</td>
<td>1.11</td>
<td>1781</td>
<td>1.11</td>
<td>84.1</td>
<td>8142</td>
<td>8151</td>
<td>.211</td>
</tr>
<tr>
<td>Australia</td>
<td>8112</td>
<td>.58</td>
<td>.671</td>
<td>.584</td>
<td>.518</td>
<td>.511</td>
<td>.511</td>
<td>.177</td>
<td>.168</td>
<td>.82</td>
<td>.11</td>
</tr>
<tr>
<td>France</td>
<td>5614</td>
<td>5167</td>
<td>6611</td>
<td>61.7</td>
<td>1412</td>
<td>1581</td>
<td>1846</td>
<td>1841</td>
<td>1228</td>
<td>1251</td>
<td>1561</td>
</tr>
<tr>
<td>UK</td>
<td>51181</td>
<td>1567</td>
<td>1157</td>
<td>1115</td>
<td>1827</td>
<td>1162</td>
<td>8516</td>
<td>8114</td>
<td>.216</td>
<td>.251</td>
<td>.718</td>
</tr>
</tbody>
</table>

Figure 2: Libyan Oman road death comparison

Figure 3: Libyan Denmark road death comparison
Figure 4: Libyan Spain road death comparisons

Figure 5: Libyan Australian road death comparison
The European Union EU has recognized road safety as a key area of its “European Transport Policy”. In fact, since 1993, the European Commission has begun to launch a series of “Road Safety Action Programmes” in order to deal more effectively with road safety vehicle performance and infrastructure.

The European Union Road Federation (ERF) represents the interests of thousands of organizations and institutions working in the road sector all over Europe. The ERF offers its resources and expertise to the European institutions in carrying out policies and programmes aimed at promoting economic growth and trade relations, improving road safety, by improving the road network.

It can be seen very clearly from Table 3, the effectiveness of the EU road safety, by reading the yearly reduction and total reduction of the number of road killing each year over the span of the last ten years time. It is very clear that the reduction reached as high as 49% less of number killing to reach 28136 at 2012 down from 51052 at the year 2003; this means an excellent achievement saving a total of 22916 human being life. Again, if Libyan rate of road killing compared of the EU rate, it is clear that the Libyan rate is going to the opposite of any of the EU countries. Accordingly, Knowledge and experience should be learned from EU to reduce this trend of killing in future.
Table 4: Number of Road fatalities for European Union Countries for the years 2003-2012.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Denmark</td>
<td>234</td>
<td>363</td>
<td>333</td>
<td>306</td>
<td>206</td>
<td>206</td>
<td>303</td>
<td>422</td>
<td>440</td>
<td>361</td>
</tr>
<tr>
<td>Ireland</td>
<td>331</td>
<td>311</td>
<td>200</td>
<td>362</td>
<td>333</td>
<td>430</td>
<td>433</td>
<td>434</td>
<td>336</td>
<td>364</td>
</tr>
<tr>
<td>Spain</td>
<td>2200</td>
<td>2123</td>
<td>2224</td>
<td>2302</td>
<td>3343</td>
<td>3300</td>
<td>4132</td>
<td>4213</td>
<td>4060</td>
<td>3303</td>
</tr>
<tr>
<td>France</td>
<td>6023</td>
<td>2230</td>
<td>2333</td>
<td>2103</td>
<td>2640</td>
<td>2412</td>
<td>2413</td>
<td>3334</td>
<td>3363</td>
<td>3623</td>
</tr>
<tr>
<td>Italy</td>
<td>6263</td>
<td>6344</td>
<td>2333</td>
<td>2663</td>
<td>2333</td>
<td>2133</td>
<td>2431</td>
<td>2030</td>
<td>3360</td>
<td>3623</td>
</tr>
<tr>
<td>Malta</td>
<td>36</td>
<td>33</td>
<td>33</td>
<td>33</td>
<td>32</td>
<td>32</td>
<td>32</td>
<td>32</td>
<td>43</td>
<td>33</td>
</tr>
<tr>
<td>Portugal</td>
<td>3224</td>
<td>3433</td>
<td>3421</td>
<td>363</td>
<td>312</td>
<td>322</td>
<td>320</td>
<td>331</td>
<td>333</td>
<td>140</td>
</tr>
<tr>
<td>Finland</td>
<td>313</td>
<td>312</td>
<td>313</td>
<td>336</td>
<td>330</td>
<td>322</td>
<td>413</td>
<td>414</td>
<td>434</td>
<td>460</td>
</tr>
<tr>
<td>Britain</td>
<td>3623</td>
<td>3363</td>
<td>3336</td>
<td>3433</td>
<td>3023</td>
<td>4622</td>
<td>4331</td>
<td>3302</td>
<td>3360</td>
<td>3304</td>
</tr>
<tr>
<td>Belgium</td>
<td>3432</td>
<td>3364</td>
<td>3033</td>
<td>3063</td>
<td>3013</td>
<td>322</td>
<td>320</td>
<td>323</td>
<td>161</td>
<td></td>
</tr>
<tr>
<td>Bulgaria</td>
<td>360</td>
<td>323</td>
<td>321</td>
<td>3023</td>
<td>3006</td>
<td>3063</td>
<td>303</td>
<td>116</td>
<td>621</td>
<td>604</td>
</tr>
<tr>
<td>Germany</td>
<td>6633</td>
<td>2324</td>
<td>2363</td>
<td>2033</td>
<td>2323</td>
<td>2211</td>
<td>2324</td>
<td>3623</td>
<td>2003</td>
<td>3600</td>
</tr>
<tr>
<td>Estonia</td>
<td>362</td>
<td>310</td>
<td>310</td>
<td>402</td>
<td>336</td>
<td>334</td>
<td>33</td>
<td>13</td>
<td>303</td>
<td>31</td>
</tr>
<tr>
<td>Greece</td>
<td>3602</td>
<td>3610</td>
<td>3623</td>
<td>3621</td>
<td>3634</td>
<td>3222</td>
<td>3226</td>
<td>3423</td>
<td>3323</td>
<td>3041</td>
</tr>
<tr>
<td>Croatia</td>
<td>103</td>
<td>603</td>
<td>231</td>
<td>632</td>
<td>633</td>
<td>662</td>
<td>223</td>
<td>246</td>
<td>233</td>
<td>330</td>
</tr>
<tr>
<td>Cyprus</td>
<td>31</td>
<td>331</td>
<td>304</td>
<td>36</td>
<td>33</td>
<td>34</td>
<td>13</td>
<td>60</td>
<td>13</td>
<td>23</td>
</tr>
<tr>
<td>Latvia</td>
<td>234</td>
<td>236</td>
<td>224</td>
<td>201</td>
<td>233</td>
<td>336</td>
<td>422</td>
<td>433</td>
<td>313</td>
<td>311</td>
</tr>
<tr>
<td>Lithuania</td>
<td>103</td>
<td>124</td>
<td>113</td>
<td>160</td>
<td>120</td>
<td>233</td>
<td>310</td>
<td>433</td>
<td>436</td>
<td>303</td>
</tr>
<tr>
<td>Luxemburg</td>
<td>23</td>
<td>20</td>
<td>21</td>
<td>23</td>
<td>26</td>
<td>23</td>
<td>23</td>
<td>34</td>
<td>33</td>
<td>32</td>
</tr>
<tr>
<td>Hungary</td>
<td>3346</td>
<td>3436</td>
<td>3413</td>
<td>3303</td>
<td>3434</td>
<td>336</td>
<td>344</td>
<td>120</td>
<td>633</td>
<td>606</td>
</tr>
<tr>
<td>Holland</td>
<td>3043</td>
<td>302</td>
<td>120</td>
<td>130</td>
<td>103</td>
<td>611</td>
<td>622</td>
<td>231</td>
<td>226</td>
<td>266</td>
</tr>
<tr>
<td>Sweden</td>
<td>243</td>
<td>230</td>
<td>220</td>
<td>222</td>
<td>213</td>
<td>331</td>
<td>323</td>
<td>446</td>
<td>333</td>
<td>432</td>
</tr>
<tr>
<td>Slovakia</td>
<td>622</td>
<td>603</td>
<td>606</td>
<td>632</td>
<td>661</td>
<td>644</td>
<td>330</td>
<td>313</td>
<td>342</td>
<td>436</td>
</tr>
<tr>
<td>Romania</td>
<td>4443</td>
<td>4224</td>
<td>4643</td>
<td>4231</td>
<td>4300</td>
<td>3063</td>
<td>4136</td>
<td>4311</td>
<td>4033</td>
<td>4024</td>
</tr>
<tr>
<td>Slovenia</td>
<td>424</td>
<td>412</td>
<td>423</td>
<td>464</td>
<td>433</td>
<td>432</td>
<td>313</td>
<td>333</td>
<td>323</td>
<td>330</td>
</tr>
<tr>
<td>Czech</td>
<td>3221</td>
<td>3334</td>
<td>3436</td>
<td>3063</td>
<td>3443</td>
<td>3016</td>
<td>303</td>
<td>304</td>
<td>114</td>
<td>124</td>
</tr>
<tr>
<td>Poland</td>
<td>2624</td>
<td>2134</td>
<td>2222</td>
<td>2423</td>
<td>2233</td>
<td>2231</td>
<td>2214</td>
<td>3303</td>
<td>2333</td>
<td>3213</td>
</tr>
<tr>
<td>Austria</td>
<td>333</td>
<td>313</td>
<td>163</td>
<td>130</td>
<td>633</td>
<td>613</td>
<td>633</td>
<td>224</td>
<td>243</td>
<td>233</td>
</tr>
<tr>
<td>Total</td>
<td>23024</td>
<td>21033</td>
<td>22023</td>
<td>23033</td>
<td>23023</td>
<td>33002</td>
<td>32063</td>
<td>33023</td>
<td>30036</td>
<td>43036</td>
</tr>
<tr>
<td>Yearly Reduction</td>
<td>%−2</td>
<td>%−6</td>
<td>%−2</td>
<td>%−2</td>
<td>%−3</td>
<td>%−3</td>
<td>%−33</td>
<td>%−33</td>
<td>%−3</td>
<td>%−3</td>
</tr>
<tr>
<td>Reduction Since 2001</td>
<td>%−1</td>
<td>%−33</td>
<td>%−36</td>
<td>%−40</td>
<td>%−43</td>
<td>%−43</td>
<td>%−36</td>
<td>%−23</td>
<td>%−22</td>
<td>%−23</td>
</tr>
</tbody>
</table>

5. Safety strategy for Libyan roads

Likewise, in many if not all other countries, Libyan road safety is a subject which needs to be tackled on several fronts and with the cooperation of a number of different agencies as well as civil society. The policy makers face many challenges in trying to invent some new road user rules and regulation. These include the need to encourage better behavior by road users, to gain acceptance of traffic rules, and to instill into road users an appreciation of a shared duty to ensure the safety of others.

The main challenge to policy makers is the need to achieve the right balance between good legislation, enforcement, engineering solution, education and public awareness measures. A balanced approach which focuses on all of these areas is necessary to achieve sustained improvements in road deaths and injuries.

One of the biggest challenges which policy makers and traffic police faces in Libya is that the road accidents are inevitable and is a form of daily life. This view should strongly and constantly be challenged, by following a clear and neat road safety strategy. Furthermore, in order to achieve sustained improvements in road safety performance, constant commitment, attention and re-evaluation to all of the various aspects and items of the safety strategy are required.

7
Road fatalities and injuries can be avoided by designing and implementing measures which reduce, as far as possible, the probability of accidents occurrences. The implementation of such measures of an integrated road safety plan involving all of the agencies and focusing on all of the relevant measures has been a feature of road safety programmers in those countries that have achieved the best road safety performance such as France, Netherlands, the United Kingdom, Sweden and other UN countries.

Libya has to learn from the experience of those best performance countries. Knowing that, there is a big difference in the culture of compliance with laws and regulations between Libya, undeveloped country, and those, developed, countries. The Libyan goals should be to reduce the incident of collisions and the effects of any if do occur. This consequently, will of course, lead to a sustained reductions in deaths and injuries. This might be done by implementing the following recommendations through a defined clear timely accounted strategy program;

- Encourage a shift in road user behavior in the key areas of speeding, seat-belt wearing and driving while intoxicated. This shift in behavior can be achieved through a range of different measures, including education through different Medias raising public awareness is of a great importance in reducing road accidents.
- Good habits must be formed from an early age and for this reason the importance of road safety education should be introduced to primary and secondary schools.
- Road pavement and shoulders maintenance should be performed periodically to avoid accidents.
- Low cost and fast measures such as road signs and marking for passing and crossing…etc.
- Use of energy absorption guard rail where ever needed.
- Removal of road side hazards and maintaining a clear right of way.
- Data bank of accidents and all aspects related to, is of importance issue and should be created and maintained properly.
- The key components of the strategy system are safer roads and roadsides (infrastructure), safer speeds, safer vehicles and consequently safer passengers.

6. Conclusion

Roads in Libya are the main cause of population killing and loss of national income. The findings of this paper show that the Libyan death rate per 100,000 Population in the last two decades is one of the highest in the Middle East, North Africa, North America and Europe. Accordingly, road safety vision and a clear strategy should be applied to reduce this killing as well as the national wealth destruction.

The adoption of quantitative targets, focusing on the importance of education, technology, engineering solutions and public awareness measures, as had been done, and under progress, in many other countries, would lead to a reduction of this killing. A national target of at least a small starting percentage reduction of collisions and fatality should be investigated, suggested, approved, and implemented as well as annually monitored and revised as necessary.

This target can be achieved by the co-operation of different sectors, agencies, civil society, and universities as well as traffic police department and ministry of transportation. The process of accidents data collection is of importance at this field of research, and need to be significantly improved in Libya.

Acknowledgements

The authors wish to thank both Engineers Ahmed El Maklofe and Mawada El Lafe Faculty of Engineering, Tripoli University, Libya for helping to produce this work. The financial assistance of the National Authority for Scientific research of the Ministry of Higher Education and Scientific research of Libya is gratefully acknowledged, for supporting the Libyan National Strategy for Road Safety Project.

References

<table>
<thead>
<tr>
<th>PAPER TITLE (90 Characters Max)</th>
<th>WIM BRIDGE: REVIEW AND FUTURE IN INDONESIA</th>
</tr>
</thead>
<tbody>
<tr>
<td>TRACK</td>
<td></td>
</tr>
<tr>
<td>AUTHOR (Capitalize Family Name)</td>
<td>POSITION</td>
</tr>
<tr>
<td>Gatot SUKMARA</td>
<td>RESEARCHER</td>
</tr>
<tr>
<td>CO-AUTHOR(S) (Capitalize Family Name)</td>
<td>POSITION</td>
</tr>
<tr>
<td>Herry VAZA</td>
<td>HEAD OF INSTITUTE OF ROAD ENGINEERING</td>
</tr>
<tr>
<td>E-MAIL (for correspondence)</td>
<td><a href="mailto:gatot.sukmara@pusjatan.pu.go.id">gatot.sukmara@pusjatan.pu.go.id</a></td>
</tr>
<tr>
<td></td>
<td><a href="mailto:herry.vaza@pusjatan.pu.go.id">herry.vaza@pusjatan.pu.go.id</a></td>
</tr>
</tbody>
</table>

KEYWORDS:
Heavy load, axle load, weigh in motion, transducers, bridge

ABSTRACT:
Bridge can excessively damage by vehicles. Its amount and weight are key elements to control the traffic and it can affect bridge health. The method of counting and weighing the vehicles axle load has been developed and used in many countries by Weigh In Motion (WIM) System by attaching transducers to pavement structure. Now, researchers is trying to make bridge as transducer for counting, weighing the vehicle axle load, and monitor bridge health. This paper reviews WIM Bridge Development and discuss its future development Indonesia.
WIM BRIDGE: REVIEW AND FUTURE IN INDONESIA
Gatot Sukmara¹, Herry Vaza²
¹²Institute of Road Engineering, Agency for Research and Development
Ministry of Public Works, Indonesia
Email for correspondence: gatot.sukmara@pusjatan.pu.go.id¹, herry.vaza@pusjatan.pu.go.id²

1. INTRODUCTION
Indonesia’s road network is approximately 446,000 km including 38,569 km of urban road and 758 km of toll road (Directorate General of Highways, Ministry of Public Work, Indonesia 2012). Road network takes 90.4% from all the transportation, while sea transportation, 7% and railway 0.6%. Indonesian road transport authority requires a vehicle data system to provide information that will enable them to manage the road transport network effectively. The key factor is applied recording system on the vehicle mass data and the number of use the bridge. Weigh In Motion (WIM) System has been used to support and collect traffic data in Indonesia other than manual counting system and static weighing system. It used to support Performance Base Contract Program for 4 Metropolitan Cities in Indonesia 2014 by the Ministry of Public Work in Indonesia (IRE, 2013).

Most of WIM in Indonesia is surface mounted type where the sensors are attached on pavement and normally need replacement in 3 to 6 months caused by transducer defect.

Bridge has been constructed in Indonesia since 1971. Currently, there are at least 89,000 bridges and 82% has a length below 100m and the rest is long span bridge. For bridge spans up to 60 m, superstructure construction usually follows Directorate General of Highways Standard. They are geographically distributed in the major islands (Vaza et al, 2008; Chen and Duan, 2014). It is very promising to develop bridges as transducers to become WIM Bridge and to monitor bridge health.

2. WEIGH IN MOTION
Weigh In Motion (WIM) system is a device that measures the dynamic axle weight of a moving vehicle to estimate the corresponding static axle mass. The basic principles of WIM technology were developed in the 1950s and variable systems were developed with different sensors and transducers in 1950-1970. The United States Bureau of Public Roads (Norman and Hopkins 1952) created a mass sensor that includes a reinforced concrete platform and is attached to the surface of the pavement. A metal-plate/rubber-sheet sandwich type capacitor scale for weighing heavy vehicles and a hydraulic-capulse transducer were used in West Germany, and spring damping transducers in the Bureau of Public Roads System of Kentucky. In the early 60’s, the collecting data was done with the help of digital computers. In the late 60s and early 70s, the Australian Road Research Board (ARRB) developed a steel plate supported along its two edges and mounted flush with the road surface, and they electronically measured the strain data. The WIM system consists of two major types, the low speed system LSEMU (<15 km/h) that became a production tool in Australia in the early 80’s and the high speed system HSEMU (>15 km/h) (Koniditsiotis et al, 1995). The WIM system with the bending plate system is still used until today. AXWAY and CULWAY were developed in Australia in the middle of the 80’s (Peter, 1986). They use bridges, especially the culvert type and the strain gauge mass sensor systems as part of their system. The same goes for FASTWEIGH in the USA (Austroads - Weigh In Motion Technology, 2000).

Today, the WIM system is more advanced and easily applicable. It is supported by the computer system and integrated on the traffic information system of many countries. Table 1 shows several brand products of Weigh In Motion Systems in the world and some of them have been used in Indonesia.
<table>
<thead>
<tr>
<th>WIM System</th>
<th>Country Origin</th>
<th>Sensor Type</th>
<th>Installation</th>
</tr>
</thead>
<tbody>
<tr>
<td>IRD Bending Plate</td>
<td>Canada</td>
<td>Bending Plate</td>
<td>Permanent, flush mounted</td>
</tr>
<tr>
<td>PAT DAW100</td>
<td>Germany</td>
<td>Bending Plate</td>
<td>Semi Permanent, flush mounted</td>
</tr>
<tr>
<td>Trevor Deakin Consultant BIMS</td>
<td>United Kingdom</td>
<td>Bending Plate</td>
<td>Semi Permanent, flush mounted</td>
</tr>
<tr>
<td>Golden River</td>
<td>South Africa/ United Kingdom</td>
<td>Capacitive Pad</td>
<td>Temporary, Surface mounted</td>
</tr>
<tr>
<td>Mikros HSWIM</td>
<td>South Africa</td>
<td>Capacitive Pad</td>
<td>Semi Permanent, flush mounted</td>
</tr>
<tr>
<td>Golden River Marksman 660</td>
<td>United Kingdom</td>
<td>Capacitive Strip</td>
<td>Permanent, flush mounted</td>
</tr>
<tr>
<td>ARRB TR HSMU</td>
<td>Australia</td>
<td>Load Cell</td>
<td>Semi Permanent, flush mounted</td>
</tr>
<tr>
<td>IRD - Single Load Cell</td>
<td>Canada</td>
<td>Load Cell</td>
<td>Permanent, flush mounted</td>
</tr>
<tr>
<td>IRD - Piezoelectric</td>
<td>Canada</td>
<td>Piezo Electric Cable</td>
<td>Permanent, flush mounted (Temporary Avail)</td>
</tr>
<tr>
<td>Trevor Deakin Consultant - PIMS Series</td>
<td>United Kingdom</td>
<td>Piezo Electric Cable</td>
<td>Permanent, flush mounted</td>
</tr>
<tr>
<td>ARRB TR - CULWAY</td>
<td>Australia</td>
<td>Strain Gauge</td>
<td>Semi Permanent, within pavement culvert installed</td>
</tr>
<tr>
<td>ARRB TR - Multi-lane CULWAY</td>
<td>Australia</td>
<td>Strain Gauge</td>
<td>Semi Permanent, within pavement culvert installed</td>
</tr>
<tr>
<td>IRD - Model 6700 WIM Mat</td>
<td>Canada</td>
<td>Capacitive Pad</td>
<td>Temporary, Surface mounted</td>
</tr>
<tr>
<td>Mikros - VLM</td>
<td>South Africa</td>
<td>Capacitive Pad</td>
<td>Temporary, Surface mounted</td>
</tr>
<tr>
<td>ARRB - PCEMU</td>
<td>Australia</td>
<td>Load Cell</td>
<td>Semi Permanent, flush mounted</td>
</tr>
<tr>
<td>PAT - DAW 50</td>
<td>Germany</td>
<td>Load Cell</td>
<td>Semi Permanent, flush mounted</td>
</tr>
<tr>
<td>Transcale - AS1 Axle Scale</td>
<td>Australia</td>
<td>Load Cell</td>
<td>Semi Permanent, flush mounted</td>
</tr>
</tbody>
</table>

3. **BRIDGE LOAD TEST ACTIVITY**

Bridge inspection uses static load testing and dynamic load testing is every 5 years according to Indonesian Bridge Inspection Guideline using special tools in addition to the visual inspection every year to analyze the condition of bridge structure. Typically, strain gages sensors are attached to the beam or to the plate, than applying a static load test and record the dynamic response when the vehicles pass through the bridge in order to determine the capacity of the bridge.

Classical load tests method sample in Indonesia is described on Wiyoto Wiyono’s inspection report on the Expressway Bridge in Jakarta that is a slab on pile type of bridge (IRE, 2013). Strain gauge is attached to the steel rebar on the deck and the strain data is recorded both in static and dynamic conditions. From the testing activity, shown that there is relationship between weight of the axle and the response from the sensor data.

Figure1 shows the installation of the gauges and the result data from the sensor due to the moving load of vehicles axle on the deck are 6.79 tones ~ 22.5 μ of front axle and 17.5 tones ~ 135 μ of each of rear axles. Truck vehicles are usually used as static load and are statically weighed. The test results are shown in a graph from the oscilloscope that recorded the change of amplitude of the strain value in time series. The peak value is similar to dynamic behavior and represents every single axle that passes trough or above the sensors (Obrien et al, 2008).
IRE used modular orthotropic to replace existing deck with an optimum application to steel truss bridge system (Irawan et al, 2010). It was tested by load test method with several sensors attached on the bottom surface of the deck. The test result of the deck system shows that the deck has a linear behavior due to the vehicles with load and unload condition. Both static loading and dynamic loading have similar characteristic with Millau Bridge, France (Jacob et al, 2013). Figure 2 shows the instrumented steel deck plate of the Millau Bridge in France. Until today, both of Wiyoto Wyono’s bridge load test or experimentally modular orthotropic steel deck system load test only covers for bridge assessment with short-term data collection and temporary system attached.

Figure 2 Picture of the extensometers glued to the bottom of the longitudinal Stiffeners at mid-span of the first span (Jacob et al, 2013)

4. BRIDGE AS TRANSDUCER AND WIM BRIDGE
The basic philosophy of the bending system that is used in many weighing systems varies in terms of analysis such as one dimension (1D), two dimensions (2D) or three dimensions (3D). The analysis includes the weighing of transducers that have linear behavior when they are loaded and unloaded as required. Figure1(c) shows that the deck has a linear behavior due to the vehicle axle load passing through the transducer. In Indonesia, bridges are used as transducers for weighing in 1D analysis. The weighing system only uses load cells for weighing the mass of vehicles in static condition and regularly distributed on the road network in Indonesia (Figure 3) to control the weight of the truck.
Second dimension (2D) analysis of a bridge as a weighing transducer is based on the influence line or the change of strain measurement that occurs in the beam or the plate when the vehicles pass over the bridge. It is also calculated in relation to other parameters. The track line of the vehicles should be constrained at one same line and should be detected with a single sensor. The span length of the beam or the girder basically affects this system. The weigh-in-motion system on orthotropic steel deck with 3D analysis has been evaluated in France where they calculated the change of the line position of vehicles in sectional plane. They also used multiple sensors as strain gauge (Jacob et al, 2013). The application of weigh-in-motion system in Indonesia has never used bridges as transducers. Besides, other WIM systems have been applied in the world such as SiWIM (Slovenia), researched on Laboratoire Central des Ponts et Chaussées (LCPC) Paris-France, which can be used on several types of bridges and applied on integral concrete slab bridges and orthotropic decks (Bouteldja, 2008).

5. **TELEMETRY SYSTEM**

There are two types of methods for data collecting of the weigh-in-motion tests. Usually, WIM systems have a portable part that is mobile and semi-permanently placed on the site, the other permanent system where the stations is telemetry system. The communication and online monitoring have been tested in Indonesia (Figure 4a). It was proved that the network can transmit to data center in a large area, both inner and inter islands of Indonesia. As Indonesia’s telecommunication network was providing the backbone for data transfer and telemetry system for transmitting sensor data from location to data center (Sukma, 2013). Figure 4 shows the experimental telemetry system in the remote monitoring of bridges in Indonesia.

![Network topology on IRE](image1.png)

(a)

![Station on integral bridge](image2.png)

(b)

Figure 4 (a) Network topology on IRE (b) Station on integral bridge

6. **CALIBRATION METHOD**

Weigh-in-Motion calibration refers to the ASTM E1318-02 standard. There are several kinds of methods in WIM calibration such as using a test truck with the specific load by using traffic stream trucks of known static weighing, as
well as calibration monitoring by using traffic stream data quality control techniques. As the computer based extraction of accurate vehicle data from measurement signals in a WIM system, the signal data can evaluate by optimization and analysis to serve as a real-time application and detect vehicle data with great accuracy. Sensor calibration on the bridge site is use of the test trucks with statistical methods (Lechner et.al, 2013). The performance of the WIM from a statistical perspective whereby the 95% confidence intervals are determined for the various errors in truck characteristic measurements (Wall et.al, 2009)

The accuracy of the calibration system depends on many factors such as the specific scale technology being used, the types of traffic environments where the WIM systems are installed, and the types and stiffness of the structures which used as transducers (NCHRP, 2008). Calibration method needs to be considered including traffic movement, speed of the vehicles, vehicles mass, and environment correction factor since Indonesia has various traffic characteristics (Hanafiah, 2013).

7. FUTURE DEVELOPMENT OF WIM BRIDGE IN INDONESIA

WIM Bridge Development requires researches and studies so that the system can be easily applied in Indonesia. Reviewing the WIM Bridge system around the world by a comparative studies and identification of classification of the bridges become the important step.

There are 89,000 bridges in Indonesia including 60,000 unit (550 km length) placed on city urban network and 29,000 unit (500 km) placed on inter urban/city road network with 78% has a short-medium span of bridge type (0 to 20 m length) and 69% of bridges has a girder for the upper structure. The short-medium span bridge become priority to be identified and concern to be develop for implementation of the WIM Bridge system, both of steel bridges (46%) and concrete bridges (36%).

The systems equipment include the use of the proper sensors need to be developed. Type of the portable system or the permanent system will be made depend to the geographical conditions in Indonesia as well as data transmission and online data recording from the site to the data center to be applied in the WIM System in wide area road.

The next stage is prototyping of the WIM Bridge system and durability test of the system and compatibility to the site location. The prototype of WIM Bridge system will be placed especially in locations with large data requirements such as expressway such as North Coast of Java Corridor and city road network. Calibration of the system that has been installed will refer to the existing world standards as it need the calibrator truck to monitoring the WIM system periodically. Monitoring and maintenance program requires for evaluate performance of the prototype WIM Bridge system. As a comparison to evaluate the data from the developed prototype of the system, existing WIM system applied in Indonesia will be install on the same site location to determine the level of the accuracy and durability of the WIM bridge prototype system. All of the development stages are not separated from the general survey and characteristics evaluation of both types of vehicles, type of structures, environmental parameters and durability of the prototype that will directly accommodate the original parameters in the traffic environment and road network in Indonesia.

8. CONCLUSION

The development of Weigh in Motion with bridge as transducers in Indonesia is very promising in the future. Many bridges distributed in wide area of Indonesia geography can become the new station of Weigh In Motion System. The WIM Bridge will support the traffic monitoring that have been applied from existing WIM system. WIM Bridge can be as productive tools that will distribute in Indonesia Road Network. It can be applied for both of the new bridge structure or place on the existing compatible bridges. Research and development program needs to apply WIM Bridge system in Indonesia as well as comparative studies to established system in the world in order to produce an applicable WIM Bridge system with expected accuracy, reasonable cost and benefit to enrich traffic data in Indonesia and more future activities such as bridge structural assessments and safety, dynamic load and testing, road and urban planning, and other applications.
9. ACKNOWLEDGEMENTS

This paper was carried out under of research program of Institute of Road Engineering Indonesia and support from all division in Institute of Road Engineering is gratefully acknowledged.

REFERENCES


Obrien, E. Žnidari, A, Ojio, T (2008) Bridge Weigh-In-Motion – Latest Developments And Applications World Wide. 5th International Conference On Weigh-In-Motion. Paris


Effective Maintenance Measures of Toll Road Pavement by Private Company

**KEYWORDS:**
Maintenance, repair, toll road, private company, pavement

**ABSTRACT:**
In recent years, budget for road maintenance has been diminishing due to the financial climate. This situation has been more prominent, particularly in local governments which cannot afford to allocate their budget, even for road repair. Therefore, effective and efficient maintenance measures for road pavement have been public demand under limited budget. However, pavement damage and condition are different in each road because these depend on traffic and temperature conditions. In order to tackle these issues from paving perspective, a series of road survey was conducted to establish road asset management system at a private toll road. This paper describes the effective road maintenance measures and techniques based on the survey, with the following conclusions are being drawn from this study. In terms of maintenance measures, appropriate rehabilitation methods are proposed based on pavement deterioration and rehabilitation history. With regard to maintenance technique, the effective maintenance using Geographical Information System (GIS) is developed to control the toll road.
Effective Maintenance Measures of Toll Road Pavement by Private Company

Masahiko Iwama¹, Hiroyasu Nakamura and Tsutomu Ihara

¹NIPPO Corporation, Saitama, Japan
Email for correspondence: iwama_masahiko@nippo-c.jp

1 INTRODUCTION

In recent years, infrastructure assets such as roads and bridges have been aging in Japan. Since Japan’s economic boom boosted from 1960's to early of 1970’s, a lot of infrastructure projects were planned and completed during that period. However, the country is facing the deterioration of infrastructures built in this high economical growth period. Therefore, various issues such as repair and rehabilitation of existing assets have been discussed among industries. In addition, as budget for public works have been diminished based on economic situation, the more effective management has been public demand in Japan. In order resolve the problem, "Asset Management Construction Support Technology of the Road Pavement" aimed at effective pavement management was developed.

2 OVERVIEW OF THE SYSTEM

In general, "asset management" means to effective management of the public properties. For road industry, this means “to predict the near future situation and effectively maintain road infrastructures within limited budget.” In other words, “asset management” is general management technique to bring the greatest advantage with the minimum expense, implementing PDCA (PLAN-DO-CHECK-ACTION) cycle. The basic concept of the “asset management” for road pavement is shown in Figure 1.

![Image of the “asset management” in road pavement](image.png)

**Figure 1: Image diagram of the “asset management” in road pavement**

2.1 Planning (Plan)

The long-term maintenance plan is made based on the evaluation of present conditions and establishment of target maintenance criteria. Then the budgets necessary for all target routes are estimated and the long-term investment planning within the budget is planned. To estimate life cycle cost (LCC), the long-term road budget can be compared. In addition, this can be a tool to explain this plan for inhabitants and road users.
2.2 Repair (DO)

The maintenance/rehabilitation constructions based on the construction method established in PLAN step are carried out. After the maintenance/rehabilitation constructions are completed, the history of the construction are recorded and used for the next steps CHECK and ACTION.

2.3 Measurement & Evaluation (CHECK)

Present road surface conditions are investigated by the road surface survey vehicle and falling weight deflectometer (FWD). The items indicated surface conditions such as cracks, rutting and roughness are measured by the road surface survey vehicle. Maintenance control index (MCI) which is Japanese surface condition index, is calculated by considering the three elements. The calculation method of MCI is shown in Equation (1) to (4), and the maintenance repair standard by this MCI is shown in Table 1. MCI can be calculated by choosing the minimum values obtained from the equations shown below.

\[
MCI = 10 - 1.48C^{0.5} - 0.29D^{0.5} - 0.47\sigma^{0.2} \\
MCI = 10 - 1.51C^{0.5} - 0.3D^{0.5} \\
MCI = 10 - 2.23\sigma^{0.5} \\
MCI = 10 - 0.54D^{0.5}
\]

where \(D\) is rutting (mm) and \(C\) is cracks (%), \(\sigma\) is roughness (mm)

| Table 1: Maintenance repair standard\(^1\) |
|---|---|
| \(MCI\) | maintenance repair standard |
| Under 3 | necessary immediately |
| 3 to 4 | necessary |
| Over 5 | desirable |

The road surface survey vehicle enables to measure the crack, rutting and longitudinal roughness of pavement simultaneously or individually. The specification of the survey vehicle is shown in Table 2.

| Table 2: Outline of survey vehicle |
|---|---|
| Measuring items | Types of measuring equipment | Measuring range | Measuring interval | Measuring accuracy | Measuring speed | Measuring method |
| Crack | Laser scanning method | Width 4 m | Traveling direction 4 mm | Distinguishing above cracking width 1 mm | | |
| Rutting | Laser light-section method | Width 4 m | Traveling direction 25 cm intersection direction 10 mm | 3 mm (for intersection profile meter) | 0-85 km/h | Electronic filing to hard disk |
| Longitudinal roughness | Laser optical displacement method | Outer part wheel 1 course of traverse | Traveling direction 50 mm | 30% (for intersection profile meter) | | |
| Distance | Tire contact type range finder | Advance 1 direction | 1 mm | 0.5% (for the steel measure tape) | | |
| Front Image | Pictures taken by camera | Approximately front 30m | 10 m | - | | |
| GPS | Car navigation type | Information of position of measurement vehicle | 10 m | 10+23 m | | |
As shown in Photo 1, the measurement devices are mounted on 4 ton based vehicle, which measures the surface evenness with 2.7m interval. Distance measured at one time is usually around 180 km in a measurement, depending on the capacity of recording media. In addition, the road surface survey vehicle is implemented to be able to evaluate the bump around the manhole. During the measurement, the road surface survey vehicle runs at a normal traffic speed during daytime. The measurement equipment of crack irradiates raiser on the road surface. The image of the surface is recorded to the hard-disc drive by detecting the beam reflected from the surface. A recorded image of the surface is shown in Photo 2.

FWD measures the surface deflections using displacement gages by dropping a weight on the pavement surface. A schematic of FWD is shown in Figure 2. As shown in Photo 3, the measurement device is equipped on the vehicle and a series of measurements is carried out. The deflection measurement is conducted at the measurement position. The vehicle can measure about 80 to 100 points per day (8 hours working time per day).

In terms of structural design, it is possible to calculate the residual number of wheels to fatigue failure and the design CBR for subgrade from deflections measured by FWD. In addition, pavement layer deterioration levels can be estimated based on Young’s moduli of each layer that are numerically calculated by inverse analysis so called "back calculation". Also "response analysis" can calculate stresses and strains occurred at arbitrary points in pavement layers.

2.4 Inspection & Review (ACTION)

Firstly, data arrangements of the present road surface condition and the structure evaluation result are performed on every route. Then, the targets of management are re-examined on every route division. In order to manage the enormous data, the database should be easy to understand and user-friendly and versatile. To meet these requirements, the Geographic Information System (GIS) is applied as an effective tool. Geographic information can be obtained through the Internet. In addition, geographic data downloaded through the Internet and installed Microsoft Excel or PDF are available for obtaining geographic information. The GIS can visually express the patterns such as map, photograph and drawing. It can also make database precisely by adding geographical information. The database developed with the GIS technology is shown in Figure 3.
3 AN EXAMPLE OF ASSET MANAGEMENT CONSTRUCTION

3.1 Outline of Private Toll Road, “Ashinoko Sky Line” and Present Conditions of Maintenance

3.1.1 Overview of the road

In this study, Ashinoko Skyline, a toll road owned by a private company, was examined for the establishment of asset management system. The road is located in a national park between Kanagawa and Shizuoka prefecture in Japan. This road started a service as a toll road in 1962. The total length of road is 10.7 km (9.0 km for special section and 1.7km for general section) and it has two lanes in each direction. The road has a several view spots and restaurants. The road is closed to traffic at night time and under bad weather. The entire road is paved by asphalt pavement. Post miles are placed in shoulder side every 100 m interval. There is no tunnel and bridge. The daily inspection of road surface and guardrail is carried out by a patrol car every day. The photo of the road is shown in Photo 4. In addition, the periodical investigation of road surface for quantity evaluation was not carried out, thus premeditated maintenance rehabilitation was not conducted.

![Photo 4: Toll gate of Ashinoko Skyline](image)

3.1.2 Traffic Density

This road can collect the information of detail traffic density and traffic vehicle division under control by taking advantage of a tollgate existing in a route. The most of heavy vehicles are buses for sightseeing purpose through a year. The average heavy traffic rate is under 1.0% of total traffic rate.

![Figure 4: Traffic volume past two years](image)

Figure 4 shows the traffic volume past two years. As can be seen in the Figure, large vehicles excepted buses was approximately 1000 vehicles/year and approximately 3 vehicles/day. This proportion was consistent during the investigation term.
3.2 Survey of Present Condition
3.2.1 Road Surface Property Examination
The road surface survey vehicle measured road surface conditions to calculate MCI. MCI were calculated by these results measured every 100 m interval. The survey results are shown in Table 3.

<table>
<thead>
<tr>
<th>LANE</th>
<th>Crack rate (percent)</th>
<th>Rutting volume (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Maximum</td>
<td>Minimum</td>
</tr>
<tr>
<td>UP</td>
<td>71.4</td>
<td>0.9</td>
</tr>
<tr>
<td>DOWN</td>
<td>56.5</td>
<td>2.1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>LANE</th>
<th>Roughness (mm)</th>
<th>MCI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum</td>
<td>Minimum</td>
<td>Average</td>
</tr>
<tr>
<td>UP</td>
<td>4.62</td>
<td>1.32</td>
</tr>
<tr>
<td>DOWN</td>
<td>4.61</td>
<td>1.26</td>
</tr>
</tbody>
</table>

From the results, it was founded that the reason for reduction in MCI is due to crack rate, not rutting volume or roughness. The accumulated distance of each MCI is shown in Figure 5.

![Figure 5: The accumulated distance of each MCI](image)

<table>
<thead>
<tr>
<th>MCI</th>
<th>Maintenance and rehabilitation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Less than 3</td>
<td>Urgent rehabilitation is needed</td>
</tr>
<tr>
<td>Less than 4</td>
<td>Rehabilitation is needed</td>
</tr>
<tr>
<td>More than 5</td>
<td>Desirable control index</td>
</tr>
</tbody>
</table>

While most results were 5 or 6 in MCI, 18% of the section (i.e. 3,800m) was 4 in MCI where repair seemed to be needed.

3.2.2 FWD Examination
The structural evaluation was conducted by measuring FWD deflections every 100 m. The residual fatigue failure time and the design CBR for sub-grade were calculated by these results. The standard of residual fatigue failure load number is 450 per three years because the heavy traffic volume in one year is about 150. The standard of design CBR for subgrade is three percent. The structural evaluation result is shown in Table 4.

<table>
<thead>
<tr>
<th>Residual number of wheel to fatigue failure</th>
<th>%</th>
<th>Design CBR for sub-grade (percent)</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Under 450</td>
<td>16.7</td>
<td>Under 3</td>
<td>27.8</td>
</tr>
<tr>
<td>Over 450</td>
<td>83.3</td>
<td>Over 3</td>
<td>72.2</td>
</tr>
</tbody>
</table>
Figure 6 shows crack rates classified according to the criteria shown in Table 5. The crack rate about fifteen years later became more than 20%. Furthermore, the future prediction LCC was calculated by probability theory.

Table 5: Rank of classification in cracks

<table>
<thead>
<tr>
<th>Rank</th>
<th>Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rank A</td>
<td>Under 10%</td>
</tr>
<tr>
<td>Rank B</td>
<td>10%~20%</td>
</tr>
<tr>
<td>Rank C</td>
<td>20%~30%</td>
</tr>
<tr>
<td>Rank D</td>
<td>30%~40%</td>
</tr>
<tr>
<td>Rank E</td>
<td>40%~50%</td>
</tr>
<tr>
<td>Rank F</td>
<td>Over 50%</td>
</tr>
</tbody>
</table>

Figure 6: Deterioration calculation of crack rate

3.2.3 Proposal for Repair Method of Construction
The repair standards are shown below:
1. The residual fatigue failure time is 100, 450 or more.
2. The design CBR for sub-grade is 3 percents.
3. The crack rate is 25 percents.

The location where the design CBR for sub-grade is under 3 is desirable to repair by the retreading. However, it is difficult to do this construction method in the private toll road, so the cement-treated base under the asphalt pavement was chosen. The chosen repair section is shown in Figure 7. According to the follow-up survey result performed after the repair, the improvement of deflection more than about 70% at the maximum was confirmed.

Figure 7: Repair section

With regard to the repair method and point, these were determined in accordance with the procedure shown in Figure 8. Firstly, crack rate is investigated to determine whether repair should be needed, then residue $T_{ref}$ estimated CBR and Fatigue Failure Number are calculated based on site investigation.

Figure 8: Procedure for selection of repair method
3.2.4 Other Examples

1. **Substantiality of homepage**
   In cooperation with a government office, the homepage of this road links to the official homepage of the sightseeing association.

2. **Melody pavement**
   Melody pavement was constructed two years ago which locates the nearest from Tokyo as melody pavement. In order to listen to melody, drivers must drive vehicle within speed limit. Each media applauded that it was effective for safety driving. In addition, the cost efficiency was good because construction was conducted with the same time of pavement repair. The melody pavement is shown in Photo 5.

![Photo 5: Melody pavement](image)

3. **Botanical garden**
   A botanical garden was constructed so that people can enjoy walking the hiking course. At the beginning of this toll road service, it was predicted that traffic density of this road would decrease every year. However, this decrease was stopped by installing various customer service facilities such as botanical garden, restaurant, and toilet in 2006. After 2006, the number of user increased by 30,000 in one year. In addition, the income and expenditure became surplus in 2009.

4 CONCLUSION
   This paper introduced the overview, management/maintenance condition, and effective maintenance method of the private toll road, Ashinoko Skyline. In order to establish more proper maintenance method, various data collection and assign of maintenance standard will be set up based on pavement deterioration or rehabilitation history. The construction method regarding scenery engineering and safe driving will be examined in this road.

   In recent years, the method of road maintenance is changing into the private company’s management, PPP and PFI. Although Ashinoko Skyline has few heavy traffic volumes comparing to general road, the accumulation of know-how on administration and maintenance methods can support private company’s management and new business expansion. This would be contributed to the further reduction in road budget and the effective maintenance in road assets for the future.

REFERENCES
   Committee of Pavement engineering (2002), *Guide of FWD and LWD*, Japan Society of Civil Engineers.
<table>
<thead>
<tr>
<th>PAPER TITLE</th>
<th>Safety Performance Functions &amp; Safety Conscious Planning</th>
</tr>
</thead>
<tbody>
<tr>
<td>TRACK</td>
<td>Indiana - A Case Study</td>
</tr>
<tr>
<td>AUTHOR</td>
<td>Position</td>
</tr>
<tr>
<td>Muhammad Asif Iqbal</td>
<td>Manager of Transportation Engineering</td>
</tr>
<tr>
<td></td>
<td>Graduate Research Assistant (2004-2006)</td>
</tr>
<tr>
<td>CO-AUTHOR(S)</td>
<td>POSITION</td>
</tr>
<tr>
<td>E-MAIL</td>
<td><a href="mailto:miqbal@aiengineers.com">miqbal@aiengineers.com</a></td>
</tr>
</tbody>
</table>

**KEYWORDS:** Safety Performance Functions (SPFs), Crash Modification Factors (CMFs), Safety Conscious Planning, Crashes, Injury/Fatal, PDO

**ABSTRACT:**

The paper presents the safety performance functions (SPFs) and Crash Modification Factors (CMFs), with reference to safety conscious planning. Traffic safety is an important component of highway design and transportation planning. The traditional approach to safety management has been to identify and remedy existing safety problems. The current planning practice addresses safety implicitly as a byproduct of adding capacity and operational efficiency to the transportation system. State highway agencies are increasingly seeking consideration of how road safety can be proactively incorporated in the short-term and long-range transportation planning processes.

The study results showed that the newly developed (present fully calibrated) SPFs perform well as compare to the past SPFs. The calibration has been defined to address the specifics of network modeling in transportation planning. An upgrade of this database on a regular basis and adding more data is therefore needed to ensure more stability of the SPFs and also for future safety analysis. In order to implement the developed SPFs in long-term planning, a set of TransCAD-based tools is needed for data pre-handling and crash prediction for segments and intersections.
SAFETY PERFORMANCE FUNCTIONS & SAFETY CONSCIOUS PLANNING
INDIANA - A CASE STUDY

Muhammad Asif Iqbal, P.E., LEED Green Associate
Manager of Transportation Engineering, AI Engineers, Inc., Middletown, CT
Graduate Research Assistant (2004-2006) Purdue University West Lafayette, IN

INTRODUCTION

Safety conscious planning (SCP) is a proactive approach to the prevention of crashes by establishing safer transportation networks through integrating safety consideration into the transportation planning process. One of the current major concerns in predicting crashes in transportation networks is the transferability, applicability and accuracy of crash prediction models, called here Safety Performance Functions (SPF). The objectives of this study were:

- evaluation of transferability of selected existing SPFs to Indiana conditions, and
- modification and recalibration of the existing SPFs or development of new SPFs for Indiana depending on the evaluation results.

The types of road facilities considered are: rural two-lane, rural multilane, rural interstate, urban two-lane, urban multilane, and urban interstate. The flow chart shown in Figure 1 illustrates framework.

![Flow Chart of Accident Prediction Algorithm](image)

Figure 1. Flow Diagram of the Accident Prediction Algorithm for a Single Roadway Segment or Intersection
NEED FOR UPDATING EXISTING MODELS

There was a need to develop a robust model that can address these shortcomings in addition to performing the following tasks: i) The models should be capable of predicting the expected number of crashes in the planning stage when more detailed actual information is not available; ii) The process of crash prediction should become more refined as more data becomes available. Models developed in the past were frequently based on an insufficient number of observations or for regions other than the studied one. These models require calibration and validation, and this paper evaluates two methods of calibrating safety prediction models for user-defined set of network partitions. The methods are tested and evaluated for the Indiana state road network.

The current models available for Indiana employed in practice need to be update due to the following factors:

- Better data is available through TransCAD.
- Some data may be irrelevant; other data may not be readily obtained.
- Current models do not contain enough variables.
- Some are outdated.

The methodology adopted for safety analysis uses SPF for various types of facilities and CMF for various safety improvements. However, this study focused on developing CMF which were used to predict crashes. As a general note, there is no difference between CMF and accident modification factors (AMF).

Different models were analyzed, supplementing and evaluating them for Indiana in order to have a set of SPF that can be used for planning studies. The models were selected on the basis of region (geography and climate), soundness of the models, quality of the data, applicability to Indiana, completeness of report/study. A set of modified models is then evaluated for transferability after calibration for Indiana conditions.

MODIFYING THE EXISTING MODELS

Modification of the existing models was done by combining CMF with Basic Safety Performance Functions (Tarko et al, 2005). Table 1 shows the Basic Safety Performance Functions (BSPF).

The general form of the BSPF is:

\[ A = k \cdot L \cdot Q^\beta \]  \hspace{1cm} (1)

where:
- \( A \) = predicted number of crashes on a segment,
- \( L \) = length of the segment as defined in section,
- \( Q \) = annual average daily traffic as defined in section,
- \( k, \beta \) = constants for specific severity level and facility type.
The safety performance functions were calibrated using the negative binomial theory. The models were evaluated in order to check the robustness in terms of crash prediction providing confidence and evidence that the developed SPF's were valid in terms of crash prediction from a stability point of view. The most common structures of the models (SPFs) for links (segments) and nodes (intersections) are as follows:

\[ A = \exp(k)LO^a \exp(\sum_{i} \gamma_i X_i) \]  

(3)

where,
\begin{align*}
A &= \text{number of crashes in a year,} \\
L &= \text{length of the section in miles,} \\
Q &= \text{AADT of the section,} \\
X &= \text{explanatory variables,} \\
k, \beta, \gamma &= \text{constants.}
\end{align*}

A model structure equivalent to equation 3 is as follows:

\[ a = BSFP \cdot AMF_1 \cdot \ldots \cdot AMF_n \]  

(4)

where:
\begin{align*}
\alpha_i &= \text{crash frequency at a specific severity level;} \\
BSFP &= \text{basic safety performance function;} \\
BSFP &= \exp(\gamma_0)E \\
AMF_i &= \text{crash modification function } i; \\
AMF_i &= \exp[\gamma_i (X_i - M_i)], \\
M_i &= \text{average or default value of characteristic } X_i.
\end{align*}

<table>
<thead>
<tr>
<th>Facility</th>
<th>Safety Performance Functions</th>
<th>Over-dispersion parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Signalized intersection</td>
<td>( a_\mu = 0.1954 \times Q^{0.723} )</td>
<td>0.639</td>
</tr>
<tr>
<td></td>
<td>( a_{PD} = 0.1758 \times Q^{1.033} )</td>
<td>0.646</td>
</tr>
<tr>
<td>Two-way stop-controlled</td>
<td>( a_\mu = 0.234 \times Q^{1.099} )</td>
<td>0.649</td>
</tr>
<tr>
<td>intersection</td>
<td>( a_{PD} = 0.307 \times Q^{1.034} )</td>
<td>0.292</td>
</tr>
<tr>
<td>All-way stop-controlled</td>
<td>( a_\mu = 0.208 \times L \times Q^{0.604} )</td>
<td>0.420</td>
</tr>
<tr>
<td>intersection</td>
<td>( a_{PD} = 0.712 \times L \times Q^{0.592} )</td>
<td>0.430</td>
</tr>
<tr>
<td>Rural two-lane segment</td>
<td>( a_\mu = 0.107 \times L \times Q^{0.814} )</td>
<td>0.451</td>
</tr>
<tr>
<td></td>
<td>( a_{PD} = 0.634 \times L \times Q^{0.615} )</td>
<td>0.484</td>
</tr>
<tr>
<td>Rural multilane segment</td>
<td>( a_\mu = 0.105 \times L \times Q^{0.880} )</td>
<td>1.253</td>
</tr>
<tr>
<td></td>
<td>( a_{PD} = 0.603 \times L \times Q^{0.896} )</td>
<td>1.349</td>
</tr>
<tr>
<td>Urban two-lane segment</td>
<td>( a_\mu = 0.674 \times L \times Q^{0.435} )</td>
<td>1.588</td>
</tr>
<tr>
<td></td>
<td>( a_{PD} = 2.028 \times L \times Q^{1.460} )</td>
<td>1.946</td>
</tr>
<tr>
<td>Urban multilane segment</td>
<td>( a_\mu = 0.044 \times L \times Q^{0.917} )</td>
<td>1.053</td>
</tr>
<tr>
<td></td>
<td>( a_{PD} = 0.169 \times L \times Q^{0.932} )</td>
<td>1.604</td>
</tr>
<tr>
<td>Rural interstate</td>
<td>( a_\mu = 0.00048 \times L \times Q^{2.238} )</td>
<td>2.383</td>
</tr>
<tr>
<td></td>
<td>( a_{PD} = 0.0057 \times L \times Q^{0.974} )</td>
<td>2.704</td>
</tr>
</tbody>
</table>

\( a_{PD} \) = typical PDO crash frequency, in PDO crashes per year,
\( a_\mu \) = typical LTP crash frequency, in LTP crashes per year,

The general form of the over-dispersion parameter.

\[ CMF = \exp[a \cdot (X - \bar{X})] \]  

(2)

where:
\begin{align*}
CMF &= \text{crash modification factor,} \\
a &= \text{regression coefficient of the variables (slope),} \\
\bar{X} &= \text{average value for the variable.}
\end{align*}
Table 2 shows the standard error of prediction was higher for the full-form SPFs than for the BSPFs. Therefore, the CMFs derived from the existing SPFs developed for other states cannot be used to adjust Indiana BSPFs.

Table 2. Standard Error of Prediction for BSPFs and SPFs

<table>
<thead>
<tr>
<th>Facility Type</th>
<th>Standard Error of Prediction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Injury/Fatal</td>
</tr>
<tr>
<td></td>
<td>BSPF</td>
</tr>
<tr>
<td>Rural two-lane roads</td>
<td>0.76</td>
</tr>
<tr>
<td>Rural multilane roads</td>
<td>1.25</td>
</tr>
<tr>
<td>Rural interstate</td>
<td>1.78</td>
</tr>
<tr>
<td>Urban two-lane roads</td>
<td>1.05</td>
</tr>
<tr>
<td>Urban multilane roads</td>
<td>1.93</td>
</tr>
<tr>
<td>Urban interstate</td>
<td>2.54</td>
</tr>
</tbody>
</table>

Figure 2. Indiana State Road Network

The above results indicate that simple calibration by adjusting the predictions with a single calibration factor is insufficient and full calibration of all model parameters (adjusting all the regression coefficients) in the models was needed. Figure 2 and Table 3 depicts the size and density of the example state network available in TransCAD for the year 2004 used for recalibration of various facilities. For each of the six road classes, the past models were recalibrated and compared with the presently developed crash prediction models. Table
4 shows an example of the calibration results for the fatal/injury crashes for Rural Two-Lane Roads in which the present (newly developed) models performed better than the past models as they had good predictive and explanatory ability as indicated by the overdispersion factor

Table 3. Sample Size Per Facility and Crash History

<table>
<thead>
<tr>
<th>Facility</th>
<th>Sample Size</th>
<th>Total Length</th>
<th>Average Length</th>
<th>Total Crashes (No.)</th>
<th>Fatal/Injury</th>
<th>PDO</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rural Two-Lane</td>
<td>11132</td>
<td>7609.37</td>
<td>0.684</td>
<td>7518</td>
<td>15442</td>
<td></td>
</tr>
<tr>
<td>Rural Multilane</td>
<td>1313</td>
<td>851.35</td>
<td>0.648</td>
<td>1969</td>
<td>4988</td>
<td></td>
</tr>
<tr>
<td>Rural Interstates</td>
<td>307</td>
<td>769.76</td>
<td>2.51</td>
<td>1769</td>
<td>8035</td>
<td></td>
</tr>
<tr>
<td>Urban Two-Lane</td>
<td>3221</td>
<td>731.35</td>
<td>0.227</td>
<td>3492</td>
<td>7884</td>
<td></td>
</tr>
<tr>
<td>Urban Multilane</td>
<td>2727</td>
<td>666.44</td>
<td>0.244</td>
<td>6276</td>
<td>16216</td>
<td></td>
</tr>
<tr>
<td>Urban Interstates</td>
<td>473</td>
<td>401.68</td>
<td>0.849</td>
<td>3532</td>
<td>15313</td>
<td></td>
</tr>
</tbody>
</table>

Table 4. Injury and Fatal Crash Model for Rural Two-Lane Roads

<table>
<thead>
<tr>
<th>Present Model</th>
<th>Coefficient</th>
<th>Std.Error.</th>
<th>Past Model</th>
<th>Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>Constant</td>
<td>0.0002</td>
<td>0.1759</td>
<td>0.2080</td>
<td></td>
</tr>
<tr>
<td>L_L</td>
<td>1.0000</td>
<td>0.0000</td>
<td>1.0000</td>
<td></td>
</tr>
<tr>
<td>L_Q</td>
<td>1.0582</td>
<td>0.0176</td>
<td>0.6040</td>
<td></td>
</tr>
<tr>
<td>LW</td>
<td>-0.1017</td>
<td>0.0170</td>
<td>-0.0831</td>
<td></td>
</tr>
<tr>
<td>RSW</td>
<td>-0.0126</td>
<td>0.0045</td>
<td>-0.0442</td>
<td></td>
</tr>
<tr>
<td>ST_SURF</td>
<td>0.3076</td>
<td>0.0302</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>ST_COMB</td>
<td>-0.6518</td>
<td>0.0231</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>ACURV</td>
<td>0.1832</td>
<td>0.0071</td>
<td>0.2158</td>
<td></td>
</tr>
<tr>
<td>AGRAD</td>
<td>-</td>
<td>-</td>
<td>0.0528</td>
<td></td>
</tr>
<tr>
<td>Alpha</td>
<td>0.8519</td>
<td>0.2140</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

CONCLUSIONS

This study investigated if existing SPF s and corresponding CMFs developed for states other than Indiana could be applied to Indiana conditions for future SCP and to develop CMFs. The BSPFs exhibited better performance than the SPF s which indicates that the CMFs added additional noise to predictions that reduced the prediction quality. Full calibration of the present (newly developed) models performed better than the past models as they had good predictive and explanatory ability as indicated by the overdispersion factor and the statistics reported in the results. However, an upgrade of this database on a regular basis and adding more years of data is therefore needed to enhance the quality of the SPF s and also for future safety analysis. The calibration has been defined to address the specifics of network modeling in transportation planning. The crash rate may change with different network, weather condition or management strategy. So without recently-collected data, the model may not be applicable. In order to implement the developed SPF s in long-term SCP, a set of TransCAD-based tools is needed for data pre-handling and crash prediction for segments and intersections.
ACKNOWLEDGEMENTS

The Indiana Department of Transportation (INDOT) for providing the funding and all relevant data to execute this study. Also, I would like to acknowledge Professor Andrew P. Tarko and all other individuals who helped in various ways during the course of this study and finally Purdue University West Lafayette, IN for providing this opportunity.

REFERENCES


**PAPER TITLE**
(90 Characters Max)
Roadway Safety in Eastern Province of Saudi Arabia: Crash Data Evaluation and Spatial Analysis

**TRACK**
Original Scientific Research

<table>
<thead>
<tr>
<th>AUTHOR (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dr. Muhammad FARHAN</td>
<td>Assistant Professor</td>
<td>Department of Transportation Engineering University of Dammam</td>
<td>Saudi Arabia</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CO-AUTHOR(S) (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sayed FARUQUE</td>
<td>Lecturer &amp; Researcher</td>
<td>Department of Transportation Engineering University of Dammam</td>
<td>Saudi Arabia</td>
</tr>
<tr>
<td>Dr. Amr MOHAMMAD</td>
<td>Assistant Professor</td>
<td>Institute of Technology, Department of Civil Engineering, West Virginia University</td>
<td>U.S.A.</td>
</tr>
<tr>
<td>Dr. Sami OSMAN</td>
<td>Assistant Professor</td>
<td>Department of Construction Engineering University of Dammam</td>
<td>Saudi Arabia</td>
</tr>
<tr>
<td>Omar AL JABARI</td>
<td>Lecturer &amp; Researcher</td>
<td>Department of Architecture and Planning University of Dammam</td>
<td>Saudi Arabia</td>
</tr>
<tr>
<td>Dr. Abdul Hamed ALMOJIL</td>
<td>Assistant Professor</td>
<td>Department of Transportation Engineering University of Dammam</td>
<td>Saudi Arabia</td>
</tr>
</tbody>
</table>

| E-MAIL (for correspondence) | |
|----------------------------| mfarhan54@gmail.com |

**KEYWORDS:**
Traffic Safety, Data Collection, Crash Data

**ABSTRACT:**
Traffic related crashes are on the rise in Saudi Arabia. With population of over 29 million, Saudi Arabia is considered as an emerging economy facing major challenges when it comes to traffic safety in particular and transportation in general. Although there are measures in place for expansion of the transportation infrastructure to address the needs of fast growing population, there is a big demerit associated to this sharp expansion in the form of increase in traffic safety related issues. Saudi Ministry of Interior reported more than 7,000 people killed and 68,000 injured in 2011 ranking Saudi Arabia among some of the worst worldwide in traffic safety. The traffic safety issues in the country also result in distress to road users and cause and economic loss exceeding 3 billion U. S. Dollars annually. The researchers in Saudi Arabia are investing in ways to improve traffic safety conditions nationwide. This paper presents a multilevel approach to evaluating traffic crash related data. Two highway corridors including King Fahd Highway 39 kilometer and Gulf Cooperation Council Highway 42 kilometer long connecting the cities of Dammam and Khobar were selected as a study area. The data was analyzed using geographic information system.
Roadway Safety in Eastern Province of Saudi Arabia: Crash Data Evaluation and Spatial Analysis

Dr. Muhammad Farhan¹, Sayed Faruque², Amr Mohammad³, Sami Osman¹, Omar Al Jabari², Abdul Hamed Al Mojil¹

¹Assistant Professor, Department of Transportation Engineering, University of Dammam, Saudi Arabia
²Lecturer, Department of Transportation Engineering, University of Dammam, Dammam, Saudi Arabia
³Assistant Professor, Department of Civil Engineering, West Virginia University, Montgomery, USA

Email for correspondence: mfarhan54@gmail.com

1 INTRODUCTION

The research was initiated in order to evaluate the traffic safety issues on two major highways of Eastern Province (EP) of the Kingdom of Saudi Arabia (KSA). A traffic safety research group was formed in association with the University of Dammam and Saudi Aramco. The main aim of this paper is to comprehensively analyze, study, and evaluate the crash data from traffic safety perspective and to find any correlation that may exist with other collected data. Four types of data were collected including Traffic Volumes, Crashes, Travel Time, and Speed. The paper provides details on the evaluation of crash data in concert with other data types collected. The Methods section of this paper describes the data collection process for each of the data types in details, and the Analyses section will provide insight on analysis of crash data.

2 LITERATURE REVIEW

The literature review provides a summary of key research relevant to this topic in order of their publication. One of the key studies investigated the relationship of accidents to length of speed-change lanes and weaving areas on Interstate highways (Cirillo et al. 1969). The results indicate that increasing the length of weaving areas will reduce the crash rate and that increasing the length of acceleration lanes will reduce accident rates. To investigate the relationship of rural highway geometry to accident rates in the state of Louisiana, a study was conducted (Dart & Mann 1970). The study concluded that the two geometric variables having the most effect on accident rates are pavement cross slope and traffic conflicts. Characteristics of intersection accidents in rural municipalities were also investigated in a research effort (Hanna et al. 1976). The research established that a typical intersection with a given volume of traffic will have a higher accident frequency under traffic signal control than under STOP or YIELD sign control. Another study investigated the design and safety on moderate volume two lane roads (Cleveland et al. 1985). Results showed a strong influence of the geometric designs on accident experience. In addition, the relation between the road environment and curve accidents were also investigated in a scientific research effort (Mathews & Barnes 1988).

The relationship between Volume to Capacity (V/C) ratio and crash rates were also investigated (Hall & Pendleton 1989). The study established that crash rates increased as a function of V/C ratio. A separate study investigated the factors influencing speed variance and its influence on accidents (Garber and Gadiraju 1990). The results showed that a major influence on speed variance is the difference between the design speed and the posted speed limit. Another effort conducted the analysis of the frequency and duration of Freeway accidents in Seattle (Jones et al. 1991). The study focused on accident analysis and prevention used the analysis to guide management strategies that seek to reduce the traffic-related impacts of crashes. The effects of roadway geometries and environmental factors on rural freeway accident frequencies were also investigated (Shankar & Barfield 1995). The research offered insight into potential measures to counter the adverse effects of weather on highway sections with challenging geometrics. Another study investigated the role of adverse weather in Key Crash Types on limited access roadways (Khattak et al. 1998). The results indicate that for the selected crash types, drivers appear to compensate for increased injury risks.

An in-depth research effort investigated the 2001 Crashworthiness Data System to see how distracted driving can impact traffic safety (Eby and Kostyniuk 2004). The effort did a thorough review of the literature on the topic and assessed the available data. Another study conducted phase II of 100-Car Naturalistic driving study (Dingus et al. 2006). The results of 100-Car field experiment was initiated to provide an unprecedented level of detail concerning driver performance, behaviour, environment, driving context and other factors that were associated with critical incidents, near crashes and crashes for 100 drivers across a period of one year.

3 STUDY AREA

Two major highways, with a total of 81 Kilometer (KM) in length, were selected as the study area. Figure 1 shows the study area. The two highways include King Fahd Bin Abdul Aziz (KF) Highway with a 39 KM long stretch, and portion of Gulf Cooperation Council (GCC) Highway with a 42 KM long stretch. The two corridors were selected because of relatively higher number of traffic crashes and crash fatalities compared to the rest of the highways in EP.
addition, they represent a vast spectrum of land-use types ranging from rural along the GCC highway and sub-urban to urban across the KF highway. The study area also covered all the intersections and interchanges, and any intersecting roads that are located within a distance of 800 meters from the two main highways.

![Image: Study Area with KF Highway and GCC Highway](image)

Figure 1: Study Area with KF Highway and GCC Highway

2 METHODS

From the literature review it is evident that for crash data analysis research, several avenues have been explored by the researchers including the geometric conditions of the highways, traffic operations, environment, and driver behavior. Past research provides valuable insight on traffic safety research yet there is a void when it comes to performing analysis on crash data in comparison to other elements that may impact traffic safety altogether. This paper attempts to collect key traffic data elements in concert and then assesses the crash data simultaneously to identify the impacts on traffic safety. Four key categories of data were collected including traffic volumes, crashes, travel times, and speed. Intersection characteristics, pavement conditions, and traffic flow videos were also collected during the field trips. Data on red light camera violation and road user stated preference survey was also collected. However, this paper will focus on crash data analysis in comparison with four key categories of data mentioned above. Separate articles would be submitted for other data types collected as the analysis on them becomes available.

2.1 Data Collection

Four types of data were collected including the traffic volumes, crashes, travel times, and speed. The traffic volume data is helpful in evaluating the speed, delay, travel times, queue lengths, intersection turning movements, and the travel demand. For this paper, the Average Daily Traffic (ADT) was observed on the study corridor for six months period starting December 02, 2013. Figure 2 shows the study area with bands of traffic volume. Due to the spatial variations in traffic flow, the data collected was grouped into categories such as data for Exit and Entry of highways, weaving sections, main highway sections, and intersections turning movements. Since the GCC Highway is limited access, the volume there was collected at three locations including the intersection between KF highway and GCC Highway in north, at the Abkake Interchange in the central section, and at the southern end of GCC highway where it meets with KF highway again. The land use around GCC highway is mostly rural, and the three data collection locations were selected keeping in view the key interchanges with other important facilities. The data were collected using two different counter technologies including the Automatic Traffic Counters (ATC) and Radar Counters. The selection of the equipment was based on the use to get higher accuracy as much as possible. In some cases the pneumatic tubes attached to the ATC were damaged due to heavy traffic load and extreme weather conditions, and the data on those locations had to be recollected. In such cases, radars were used for the data recollection. At few locations radar equipment was mishandled by the law enforcement personnel thus making the data collected by that equipment obsolete. Some other obstacles in collection of the data include safety of road users and data collection staff.

790
Total travel time and delay were also collected on the two corridors by means of active test vehicle technique (floating cars) (Milton & Manering 1998a) equipped with video cameras and Global Positioning System (GPS). Travel times, trip delays, traffic signal delays, congestion delays and average speeds on selected segments were collected as well. The study area was divided into nineteen segments with ten segments on KF Highway and nine on the GCC Highway. The segments were selected to represent homogeneous cruising conditions. Test vehicles were then dispatched to drive with the traffic stream on the segments on AM peak direction from 6:00 AM to 9:00 AM and in PM peak direction from 3:00 PM to 6:00 PM periods. Video camera footage was recorded with the drivers’ own voice remarks. Travel time data were then extracted from the video recordings. Drivers maintained the test vehicle speed by means of "floating car" technique (Milton & Manering 1998b) in which the vehicle was driven by safely passing as many vehicles as pass the test vehicles.

Crash Data Analysis with respect to Speed was an imperative input to this paper as the literature suggests that speed impacts traffic safety (Taylor et al. 2002). Speed data were collected from 88 sites, on at least 150 vehicles at each collection site along the KF and GCC Highways. Locations and times chosen for speed data collection were based on crash data and traffic volumes. Different technologies, including fixed and handheld radars, and pneumatic road tubes were used for the data collection.

2.2 Crash Data

Crash data was collected to identify high crash risk areas and to find a correlation between the collected data with respect to traffic safety. Crash data collection in KSA is done by Traffic Police. The traffic police collect the data for internal use only and the data as such is not intended primarily for traffic safety research use. For this paper, the crash data was collected from the Police. The data was then processed by the staff at the University of Dammam. The data was grouped into four classes according to severity including Fatal, Major, Minor, and Damage only. Table 1 describes the crash data collected by year and by four classes.

Table 1: Crash Data Collected from the EP Police

<table>
<thead>
<tr>
<th>Year</th>
<th>Fatal</th>
<th>Serious</th>
<th>Minor</th>
<th>Damage Only</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>2008</td>
<td>1</td>
<td>23</td>
<td>0</td>
<td>1</td>
<td>25</td>
</tr>
<tr>
<td>2009</td>
<td>276</td>
<td>1157</td>
<td>3</td>
<td>137</td>
<td>1573</td>
</tr>
<tr>
<td>2010</td>
<td>213</td>
<td>1132</td>
<td>7</td>
<td>406</td>
<td>1758</td>
</tr>
<tr>
<td>2011</td>
<td>232</td>
<td>1161</td>
<td>20</td>
<td>10156</td>
<td>11569</td>
</tr>
<tr>
<td>2012</td>
<td>346</td>
<td>2088</td>
<td>76</td>
<td>18026</td>
<td>20536</td>
</tr>
<tr>
<td>2013</td>
<td>225</td>
<td>1887</td>
<td>77</td>
<td>4954</td>
<td>6943</td>
</tr>
<tr>
<td>2014</td>
<td>18</td>
<td>86</td>
<td>10</td>
<td>1</td>
<td>113</td>
</tr>
<tr>
<td>Total</td>
<td>1300</td>
<td>7334</td>
<td>193</td>
<td>33681</td>
<td>42517</td>
</tr>
</tbody>
</table>

Table 1 shows a large variation in "damage only" category. On further investigation, it was revealed that the "damage only" group had data entry discrepancies originating at source and re-collecting the data or data correction would not be possible at source. It was therefore decided to drop the damage only group from the analysis for this paper. The EP
Police department started the data collection in the mid of 2008 and the collection work for the year 2014 was not completed as yet. Therefore, the data for the year 2008 and 2014 were filtered out as well. From the perspective of geo-spatiality, the raw data was grouped depending on the availability of geo-coordinates. Table 2 shows the crash data with or without geo-coordinates.

Table 2: Crash data with and without geo-coordinates

<table>
<thead>
<tr>
<th>Year</th>
<th>Geo-coded</th>
<th>Non-Geo-coded</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>2008</td>
<td>2</td>
<td>23</td>
<td>25</td>
</tr>
<tr>
<td>2009</td>
<td>318</td>
<td>1255</td>
<td>1573</td>
</tr>
<tr>
<td>2010</td>
<td>430</td>
<td>1328</td>
<td>1758</td>
</tr>
<tr>
<td>2011</td>
<td>2992</td>
<td>8577</td>
<td>11569</td>
</tr>
<tr>
<td>2012</td>
<td>6366</td>
<td>4170</td>
<td>20536</td>
</tr>
<tr>
<td>2013</td>
<td>6350</td>
<td>593</td>
<td>6943</td>
</tr>
<tr>
<td>2014</td>
<td>80</td>
<td>33</td>
<td>113</td>
</tr>
<tr>
<td>Total</td>
<td>20538</td>
<td>15979</td>
<td>42517</td>
</tr>
<tr>
<td>Percentage</td>
<td>62</td>
<td>38</td>
<td>100</td>
</tr>
</tbody>
</table>

Table 2 also shows that 38% of all the collected crash data were without geo-coordinates. This could be because either at the crash scene the forms were not completed appropriately or the global positioning device at the location malfunctioned. The data without geo-coordinates could not be allocated to any locations in the study area. Even if the writers try to get the location of the data by address mentioned in the form, the crash data still cannot be allocated at the exact crash location. The data without geo-coordinates was therefore discarded. The following points were inferred from crash data cleaning process:

1. Only fatal, major, and minor injuries data seemed plausible due to the inconsistency with damage only data.
2. Data from 2008 and 2014 were incomplete as the data collection started in the mid of 2008 and 2014 is on-going.
3. Crash data with no geo-coordinates was filtered as it cannot be allocated.

Table 3 shows the final output of data cleaning process by year and by crash severity.

Table 3: Filtered Crash Data by Severity and by Year

<table>
<thead>
<tr>
<th>Severity</th>
<th>Fatal</th>
<th>Serious</th>
<th>Minor</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>2009</td>
<td>82</td>
<td>216</td>
<td>2</td>
<td>300</td>
</tr>
<tr>
<td>2010</td>
<td>64</td>
<td>246</td>
<td>2</td>
<td>312</td>
</tr>
<tr>
<td>2011</td>
<td>62</td>
<td>382</td>
<td>5</td>
<td>449</td>
</tr>
<tr>
<td>2012</td>
<td>222</td>
<td>1381</td>
<td>64</td>
<td>1667</td>
</tr>
<tr>
<td>2013</td>
<td>170</td>
<td>1377</td>
<td>63</td>
<td>1610</td>
</tr>
<tr>
<td>Total</td>
<td>600</td>
<td>3602</td>
<td>136</td>
<td>4338</td>
</tr>
</tbody>
</table>

It is also noted that out of the total 42,517 crash records from Table 1 only 4,338 crash records shown in Table 3 were useable. The rest of the data were discarded due to inconsistencies, one of the limitations of this paper.

3 ANALYSES

Traffic volume data was populated on the study area highway network and was overlaid with the crash data to see any correlation. Figure 3 shows the traffic volume bandwidth overlaid with crash data grouped in fatal injury and major injury category only. It was noted that the crashes with major injuries existed all along the KF and GCC highways. Fatal accidents were higher at segments with higher traffic volumes. A number of fatal crashes did show up on the collector facilities where the traffic volume data was not collected. Higher volume bandwidth area of Abkake Interchange on GCC highway had highest number of fatal crashes compared to the rest of the study area. It was also noted that low volume segments on KF highway had fewer fatal crashes.
The crash data was evaluated for the causes reported with the data. Figure 4 describes the proportions of 10 of the top causes reported including driver distraction, speeding, red light running, illegal overtaking, not giving way, sudden turning, insufficient distance, pedestrian crossing, violating pedestrian sign, and others.

The pie chart shows that the driver’s distraction at 26% is one of the major categories for crashes. Speeding at 14% and speed related issues (Red light running at 3% and sudden tuning at 26%) sum up to be the biggest cause totaling 43%. Geo-spatial method to evaluate the visual trends of road crash density was also applied over the study area. PTV Software VISUM Safety version 13.0 was used to develop the heat maps based on the data starting 2009 to 2013. To understand the variability of crash clustering on heat maps, a direct count method within a search radius of 500 meter was applied. The number of crashes “n” within the search radius defined the crash risk level. The following criteria were used:

a. Low risk area with n< 5
b. Moderate risk area with 5<n<10
c. High risk area with n≥ 10

Figure 5 part a), part b), part c), part d), and part e) show crash densities in the study area for the years 2009 to 2013 respectively. The red represents the high crash density, the yellow is moderate, and green is low crash density.
Figure 5: Heat Map of Crash Densities part a) 2009, part b) 2010, part c) 2011, part d) 2012, and part e) 2013

Figure 5 part a) to part c) describe that the crash densities do not vary much from 2009 to 2011. Figure 5 part d) and part e) however show several crash density areas where the high crash densities spread all over. With improvements to crash data collection process with respect to inconsistencies we noted, the data in Figure 4 can be further enhanced.

The travel time data gave clues on the problematic locations with delays in peak directions of traffic flow. The spatial information on AM delay was overlaid on GIS map of the two corridors with crash data. Figure 6 part a) and part b) show the delay overlaid with overall crash data and speed related crash data respectively. The results show more crashes in low delay areas compared to the areas with high delay as seen in part a). Also part b) shows that the high speed segment of the corridor had higher number of crashes compared to the low speed segment of the corridor with relatively higher delays. It was also observed from Figure 6 part b) that the numbers of fatal crashes were lower in high delay areas when compared to the areas with high speed.

Figure 6: Part a) Delay overlaid with Crash Data Part b) Delay overlaid with Speed Related Crash Data
Spot speed studies were used to determine essential statistical measures such as speed distributions, the pace, median speeds and percentiles. The analysis was carried out for the speed data collected under prevailing flow conditions from the 88 sites across the two corridors. Results show variations of 85th percentile speeds at different locations of the "segments with same posted speed limits". In addition, the study showed an average of 20% rate of speed limit "noncompliance" along the two corridors. Figure 8 describes the speed data showing percent difference between posted speed and observed speed overlaid with the crash data showing fatal and major injuries. From Figure 7, it is clear that speed noncompliance areas are the fatal and major crash high risk zones.

4 CONCLUSIONS

The concerted effort of collecting the traffic safety related data is powerful when the data is overlaid using geographic information system for safety analysis. The data such as speed, traffic volume, and travel time when overlaid and compared with crash data show clear correlation. The segments with higher speeds showed more fatal crashes compared to the ones with higher delays. The crashes with major injuries occurred all along the corridors more so at segments with higher bands of traffic volume. It was also noted that pre-collected crash data may not be suitable for comprehensive traffic safety studies, especially in countries where infrastructure is still developing and the data may not have been collected with traffic safety research in mind. In-depth crash data collection would be the best approach to address the limitations of inconsistencies in crash data for traffic safety studies. We recommend increasing the public awareness campaigns that target speeding issues, increasing the level of enforcement and the development of a traffic operation center for better traffic safety related data collection and analysis.

5 ACKNOWLEDGEMENTS

The key partner agencies included the University of Dammam, Saudi Aramco, the Traffic Police, the Ministry of Transport, and the Municipalities of Dammam.

6 REFERENCES


PAPER TITLE
(90 Characters Max)
Modern Roundabout Safety Assessment in the United States

TRACK
Road safety

AUTHOR
(Capitalize Family Name)
Aemal KHATTAK
Associate Professor
University of Nebraska-Lincoln
United States

CO-AUTHOR(S)
(Capitalize Family Name)

POSITION

ORGANIZATION

COUNTRY

E-MAIL
(for correspondence)
aemalkhattak@gmail.com

KEYWORDS:
Safety, Roundabouts, Accidents, Crash Data Analysis, Safety at Intersections, Two-Way Stop-Controlled Intersections, Highway Safety Manual

ABSTRACT:
The focus of this paper is on the safety assessment of conversion of two-way stop-controlled intersections to modern roundabouts in the United States using guidelines available in the Highway Safety Manual, which was published by the American Association of State Highway Transportation Officials (AASHTO) in 2010. After review of pertinent literature, a case study analyzing the safety improvement resulting from conversion of two-way stop-controlled intersections to roundabouts in Kansas is presented. Crash and traffic data on four intersections that were converted to roundabouts in Kansas were obtained from the Kansas Department of Transportation. Analysis showed that all types of crashes including fatal, non-fatal injury and property-damage-only (PDO) were reduced after conversion to roundabouts. Total crashes reduced by 58.13% on the whole. Fatal and non-fatal injury crashes reduced by 100% and 76.47%, respectively while PDO crashes were reduced by 35.49%. Overall, the conversion to roundabouts significantly improved public safety.
Modern Roundabout Safety Assessment in the United States

Dr. Aemal Khattak
University of Nebraska-Lincoln, Lincoln, Nebraska, USA
Email for correspondence: aemalkhattak@gmail.com

1 INTRODUCTION

Intersections where two or more roadways cross at the same level are critical junctions in the transportation network as the safety and efficiency of the road network depends on these junctions to a significant extent. Traffic control is an important consideration at intersections due to competing needs of the conflicting traffic. The usual traffic control in the United States (US) includes use of yield signs on minor approaches, stop signs on minor approaches, stop signs on all approaches, and signalization. An alternative to these is the modern roundabout, construction of which is becoming popular in the US. The use of roundabouts in the US began in 1990s and has been increasingly popular since then (Rodegerdts 2007). Studies in the US have shown that roundabouts are effective in urban environments but relatively little published literature is available on the safety performance of roundabouts constructed on high-speed (72+ kmph, 40+ mph) roads in rural and suburban areas.

A concern with roundabouts constructed on high-speed rural roadways is the speed differential of vehicles traveling on the roundabout approaches and roundabout entries. Roundabouts on high-speed roadways are not “high-speed roundabouts” (Isebrands and Hallmark 2012). With a well-designed roundabout, drivers are allowed to navigate at a reduced speed (24-48 kmph, 15-30 mph) inside the roundabout (Isebrands and Hallmark 2012, Persaud et al. 2001, Rodegerdts 2010). Inadequate signing, absence of nighttime lighting and possible lower levels of drivers’ alertness in rural environments may be some of the reasons causing high approach speeds and driver confusion at the roundabouts (Thomas and Nicholson 2003, Appleton and Clark 1998). The research question addressed in this paper was: “Are roundabouts on high-speed roadways in the US safer than two-way stop-controlled (TWSC) intersections?” As such, the objective was to statistically quantify the changes in reported accidents before and after construction of rural high-speed TWSC roundabouts.

To address the above research question, accident records on four TWSC intersections that were subsequently converted to roundabouts were collected from Kansas Department of Transportation (KDOT). A before-after analysis using the Empirical Bayes (EB) method as given in the Highway Safety Manual was utilized. A review of pertinent literature follows this introduction after which results of the EB before-after accident analysis are presented. The last section provides a summary and conclusions.

2 LITERATURE REVIEW

Published studies focused on the safety of rural high-speed approach roundabouts were reviewed and the following studies were uncovered on rural high-speed roundabouts. Myers (1999) studied accidents at five high-speed rural intersections in Maryland that were converted to roundabouts. Analysis showed that the average accident rate at these intersections after conversion to roundabouts reduced by 59% and that injury or serious accidents were reduced by 80%. Persaud et al. (2001) conducted an EB observational before-after study of accidents when twenty-three intersections were converted from stop sign and traffic signal control to modern roundabouts. Results showed a 40% reduction in all accidents and an 80% reduction in injury accidents. Of all the intersections, the five rural single-lane roundabouts experienced a 58% reduction in total accidents and an 82% reduction in injury accidents, which were both higher than the average of all settings. Richie and Lenters (2005) compared the performance of roundabouts and traffic signals on high-speed approaches (45+ mph). They reported roundabouts resulting in nearly a 50% reduction in injury and fatal accidents compared to signalized intersections; one specific site demonstrated an 80% reduction in expected accidents after it was converted to a roundabout.

Rodegerdts (2007) conducted an EB before-after study comparing the performance of traditionally controlled intersections with roundabouts. For the 9 rural roundabouts converted from TWSC intersections, the total accident reduction was 71.5 % and injury accidents were reduced by 87.3 %. For 24 suburban roundabouts converted from signalized or TWSC intersections, the total accident reduction and injury accidents reduction were about 42% and 68%, respectively. In Maryland, accident reports of 149 crashes at twenty-nine single-lane roundabouts and 134 accidents at nine double-lane roundabouts were reviewed (Mandavilli et al. 2009). Several of the roundabouts in the study were rural roundabouts on high-speed roadways; about three quarters of all reported collisions were at roundabout entrance and high approach speed was an important factor in accidents.

Isebrands and Hallmark (2012) conducted a study on rural roundabouts with high-speed approaches. The before-after EB estimation showed reductions of 67% in total accidents and 87% in injury accidents. The study reported that speed reduction before roundabouts on rural roadways was greater than at stop-controlled approaches. An
evaluation of conversion to roundabouts was conducted in Wisconsin (Qin et al. 2013). The EB before-after analysis of eight rural roundabouts showed reductions of 45% in total accidents and 56% in fatal and injury accidents. The study included 11 roundabouts with posted speed limit of 45 mph or greater (72 kmph or greater). These roundabouts experienced reductions of 34% in total accidents and 49% in fatal and injury accidents.

A study of conversion to roundabouts in Belgium (Antoine 2005) showed an average of 42% decrease in injury accidents and 48% decrease in serious accidents in all settings. Roundabouts in rural open country environment, which usually have high speed approaches, had a 50% accident reduction. Roundabouts in suburban locations had an accident reduction of 46% and those in urban areas a reduction of 15%.

In summary, the reviewed literature showed that roundabouts converted from traditionally controlled intersections with high-speed approaches in rural and suburban areas improved safety. However, more research is still needed before we draw the conclusion that roundabouts are the most appropriate control for intersections with high-speed approaches in rural settings. This study therefore explored the safety performance of high-speed roundabouts in rural settings, using data obtained from Kansas; the studied roundabouts were all TWSC intersections before conversion.

3 EMPIRICAL BAYES BEFORE-AFTER ANALYSIS

The EB before-after analysis is a safety effectiveness evaluation method that uses Safety Performance Functions (SPFs) to estimate what the expected average accident frequency would have been at a location where a safety improvement treatment was applied, had the treatment not been implemented. It then compares the actual observed accidents after treatment application to the expected average if the treatment had not been applied to determine the treatment’s safety effectiveness (AASHTO 2010).

The fluctuation of accidents over time at a location makes it difficult to determine whether changes in accident frequencies are due to a safety treatment or due to the fluctuation. For example, when a site experiences a high accident frequency in a certain period, it is statistically probable that it will experience a comparatively low accident frequency in the following period of similar duration. This phenomenon is known as regression-to-the-mean (RTM). Compared to simple before-after analysis, EB results are adjusted by changes in traffic volumes and corrected for potential biases from the RTM effect. The EB method is used in the Highway Safety Manual (AASHTO 2010). The procedures are described as follows.

The predicted average accident frequency for a year, \( N_{predicted} \), is expressed as per intersection per year is:

\[
N_{predicted} = N_{spf} \times (CMF_{1x} \times CMF_{2x} \times \ldots \times CMF_{yx}) \times C_x
\]  

Where

- \( N_{spf} \) = predicted average accident frequency determined for base condition of the Safety Performance function (SPF) developed for site type \( x \),
- \( CMF_{yx} \) = accident modification factors specific to SPF for site type \( x \), and
- \( C_x \) = calibration factor to adjust SPF for local conditions for site type \( x \).

The expected average accident frequency for the before treatment period is expressed as per intersection summed for the entire before period.

\[
N_{expected,B} = w_{i,B}N_{predicted,B} + (1 - w_{i,B})N_{observed,B}
\]

Where, the weight for each site \( i \) is determined as:

\[
w_{i,B} = \frac{1}{1+k \sum_{before\ years} N_{predicted}}
\]

\( N_{expected,B} \) = Expected average accident frequency at site \( i \) for the entire before treatment period,

\( N_{observed,B} \) = Observed accident frequency at site \( i \) for the entire before treatment period, and

\( k \) = over-dispersion parameter for the applicable SPF.

The predicted average accident frequency for each site \( i \) during each year of the after treatment period can be calculated in the same way. The adjustment factor, \( r_i \), which accounts for the difference between the before and after treatment periods in duration and traffic volume at each site \( i \) is:

\[
r_i = \frac{\sum_{after\ years} N_{predicted,A}}{\sum_{before\ years} N_{predicted,B}}
\]

Then the expected average accident frequency for each site \( i \) over the entire after period in the absence of the treatment is:

\[
N_{expected,A} = N_{expected,B} \times r_i
\]

The estimate of the safety effectiveness of the treatment at site \( i \) can be expressed in the form of an odds ratio,

\[
OR_i = \frac{N_{observed,A}}{N_{expected,A}}
\]

The percentage accident change at site \( i \) is:

\[
P_i = 100 \times (1 - OR_i)
\]

The overall effectiveness of the treatment for all sites combined, in the form of an odds ratio, is expressed as:

\[
OR' = \frac{\sum_{all\ sites} N_{observed,A}}{\sum_{all\ sites} N_{expected,A}}
\]

799
The odds ratio above is potentially biased. An unbiased estimate of the overall effectiveness is:

\[
OR = \frac{\text{OR}}{1 + \frac{\text{Var} \left( \sum_{\text{sites}} N_{\text{expected}, A} \right)}{\left( \sum_{\text{sites}} N_{\text{expected}, A} \right)^2}}
\]

(9)

In which, \(\text{Var} \left( \sum_{\text{sites}} N_{\text{expected}, A} \right) = \sum_{\text{sites}} \left( r_i \right)^2 \times N_{\text{expected}, A} \times (1 - w_i)^2\).

4 ACCIDENT DATA AND ANALYSIS

Accident data on four rural high-speed (72-107 kmph, 45-65 mph) intersections with two-way stop control that were converted to roundabouts were obtained from KDOT. The period when two-way stop control was in effect was referred to as the “before” time period (i.e., before conversion to roundabouts) while the roundabout period was termed as the “after” period; conversion to roundabout was the safety treatment in each case. Accidents reported during the conversion year were excluded to remove any construction effects. Information for fatal, injury and property-damage-only (PDO) accidents for each year in the before and after periods was utilized in the analysis. Table 1 presents the locations of the four roundabouts, the accident counts in the two time periods and annual average daily traffic (AADT) before and after roundabout conversion.

Table 1. Information on the four intersections converted to roundabouts

<table>
<thead>
<tr>
<th>Intersecting Roads</th>
<th>Conversion Year</th>
<th>Number of Legs</th>
<th>Before Period</th>
<th>After Period</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Years</td>
<td>Total accidents</td>
</tr>
</tbody>
</table>

Table 2 presents the results of the EB before-after analysis for total, fatal, non-fatal injury, and property-damage-only (PDO) accidents reported at each site. The odds ratios (column 5) values smaller than 1.00 indicate that fewer accidents were reported after conversion to roundabouts. Percentage reductions (column 6) represent accident reduction rates; larger values represent greater accident reductions. The intersection at US-400 & K-47 experienced an increase in total accidents after conversion, a 100% decrease in fatal accidents and a slightly decrease in injury accidents. The other three locations had a percentage reduction ranging from 45% to 84% for total accidents, 100% for fatal accidents and from 80% to 100% for injury accidents. The results for the PDO accidents, however, were mixed as two locations experienced an increase in such accidents.

Table 3 presents the results of aggregated analysis of all four locations, i.e., accidents at all locations in each time period were pooled for the analysis. The overall effectiveness of the treatment (conversion to roundabouts) for all sites combined can be expressed in the form of an odds ratio (column 5). This odds ratio is potentially biased but an unbiased estimate of the overall effectiveness is presented in column 6. Overall, all types of accidents were reduced after conversion to roundabouts. Total accidents were reduced by 58.13%; fatal accidents were reduced by 100%; injury accidents were reduced by 76.47% while property-damage-only accidents were reduced by 35.49%. The results are mostly consistent with other studies reported in the literature.
Table 2. Empirical Bayes analysis of all accidents

<table>
<thead>
<tr>
<th>Intersecting Roads</th>
<th>Observed Total Accidents (Before)</th>
<th>Observed Total Accidents (After)</th>
<th>Expected Total Accidents (After)</th>
<th>Odds Ratio (Observed/Expected)</th>
<th>Percentage Reduction % [100*(1-Odds Ratio)]</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Total Accidents</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>US-400 &amp; K-47</td>
<td>21.00</td>
<td>9.00</td>
<td>8.03</td>
<td>1.12</td>
<td>-12.10</td>
</tr>
<tr>
<td>US-400/US-69A &amp; K-66</td>
<td>19.00</td>
<td>3.00</td>
<td>19.34</td>
<td>0.16</td>
<td>84.48</td>
</tr>
<tr>
<td>E. Jct. of US-77 &amp; US-166</td>
<td>21.00</td>
<td>3.00</td>
<td>13.64</td>
<td>0.22</td>
<td>78.01</td>
</tr>
<tr>
<td>US-50 &amp; US-77</td>
<td>20.00</td>
<td>9.00</td>
<td>16.31</td>
<td>0.55</td>
<td>44.81</td>
</tr>
<tr>
<td><strong>Fatal Accidents</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>US-400 &amp; K-47</td>
<td>3.00</td>
<td>0.00</td>
<td>0.36</td>
<td>0.00</td>
<td>100.00</td>
</tr>
<tr>
<td>US-400/US-69A &amp; K-66</td>
<td>0.00</td>
<td>0.00</td>
<td>0.41</td>
<td>0.00</td>
<td>100.00</td>
</tr>
<tr>
<td>E. Jct. of US-77 &amp; US-166</td>
<td>0.00</td>
<td>0.00</td>
<td>0.40</td>
<td>0.00</td>
<td>100.00</td>
</tr>
<tr>
<td>US-50 &amp; US-77</td>
<td>3.00</td>
<td>0.00</td>
<td>0.43</td>
<td>0.00</td>
<td>100.00</td>
</tr>
<tr>
<td><strong>Non-fatal Injury Accidents</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>US-400 &amp; K-47</td>
<td>10.00</td>
<td>4.00</td>
<td>4.26</td>
<td>0.94</td>
<td>6.15</td>
</tr>
<tr>
<td>US-400/US-69A &amp; K-66</td>
<td>10.00</td>
<td>0.00</td>
<td>9.31</td>
<td>0.00</td>
<td>100.00</td>
</tr>
<tr>
<td>E. Jct. of US-77 &amp; US-166</td>
<td>11.00</td>
<td>1.00</td>
<td>6.57</td>
<td>0.15</td>
<td>84.78</td>
</tr>
<tr>
<td>US-50 &amp; US-77</td>
<td>11.00</td>
<td>2.00</td>
<td>9.60</td>
<td>0.21</td>
<td>79.17</td>
</tr>
<tr>
<td><strong>Property-damage-only (PDO) Accidents</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>US-400 &amp; K-47</td>
<td>8.00</td>
<td>5.00</td>
<td>3.60</td>
<td>1.39</td>
<td>-38.99</td>
</tr>
<tr>
<td>US-400/US-69A &amp; K-66</td>
<td>9.00</td>
<td>3.00</td>
<td>10.02</td>
<td>0.31</td>
<td>68.85</td>
</tr>
<tr>
<td>E. Jct. of US-77 &amp; US-166</td>
<td>10.00</td>
<td>2.00</td>
<td>10.84</td>
<td>0.29</td>
<td>70.69</td>
</tr>
<tr>
<td>US-50 &amp; US-77</td>
<td>6.00</td>
<td>7.00</td>
<td>6.82</td>
<td>1.11</td>
<td>-11.04</td>
</tr>
</tbody>
</table>
Table 3. Empirical Bayes before-after analysis for all locations (aggregated)

<table>
<thead>
<tr>
<th>Accident Type</th>
<th>Observed Accidents (After)</th>
<th>Expected Accidents (After)</th>
<th>Percentage Change %</th>
<th>Odds Ratio</th>
<th>Unbiased Odds Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total</td>
<td>24.00</td>
<td>57.31</td>
<td>158.13</td>
<td>0.42</td>
<td>0.41</td>
</tr>
<tr>
<td>Fatal</td>
<td>0.00</td>
<td>1.22</td>
<td>100.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Injury</td>
<td>7.00</td>
<td>29.74</td>
<td>76.47</td>
<td>0.24</td>
<td>0.23</td>
</tr>
<tr>
<td>Property-damage-only</td>
<td>17.00</td>
<td>26.35</td>
<td>35.49</td>
<td>0.65</td>
<td>0.63</td>
</tr>
</tbody>
</table>

Table 4 presents the before-after analysis of fatality and injury rates at the four locations. Fatality and injury rates (on a per-year base) in all four locations were reduced after conversions to roundabouts. Fatality rates were reduced by 100% while injury rates were reduced by at least 60%. The analysis showed that severe accidents significantly decreased after the TWSC intersections were converted to roundabouts.

Table 4 Before-after analysis of death and injury rates (per year)

<table>
<thead>
<tr>
<th>Location</th>
<th>Death Rate (Before)</th>
<th>Injury Rate (Before)</th>
<th>Death Rate (After)</th>
<th>Injury Rate (After)</th>
<th>Death Rate Change %</th>
<th>Injury Rate Change %</th>
</tr>
</thead>
<tbody>
<tr>
<td>US-400 &amp; K-47</td>
<td>0.60</td>
<td>5.80</td>
<td>0.00</td>
<td>2.33</td>
<td>-100.00</td>
<td>-59.77</td>
</tr>
<tr>
<td>US-400/US-69A &amp; K-66</td>
<td>0.00</td>
<td>4.75</td>
<td>0.00</td>
<td>0.00</td>
<td>-</td>
<td>-100.00</td>
</tr>
<tr>
<td>E. Jct. of US-77 &amp; US-166</td>
<td>0.00</td>
<td>5.00</td>
<td>0.00</td>
<td>0.33</td>
<td>-</td>
<td>-93.33</td>
</tr>
<tr>
<td>US-50 &amp; US-77</td>
<td>1.00</td>
<td>7.00</td>
<td>0.00</td>
<td>0.50</td>
<td>-100.00</td>
<td>-92.86</td>
</tr>
<tr>
<td>All Sites</td>
<td>0.39</td>
<td>5.61</td>
<td>0.00</td>
<td>0.71</td>
<td>-100.00</td>
<td>-87.27</td>
</tr>
</tbody>
</table>

5 CONCLUSIONS

This study focused on the assessment of four rural high-speed TWSC intersections that were converted to roundabouts in Kansas. The evaluation procedures utilized were from the Highway Safety Manual (AASHTO 2010). Analysis results showed that overall all types of accidents were reduced after roundabout conversion. Total accidents decreased by 58.13%; fatal accidents were reduced at 100% at all locations and non-fatal injury accidents were reduced with an overall reduction rate of 76.47%. Property-damage-only accidents were reduced by 35.49% as a whole but two out of the four sites experienced an increase in property-damage-only accidents after conversion to roundabouts. Based on a before-after analysis, fatality and injury rates were found decreased at all four sites. In conclusion, there is statistically significant evidence that modern roundabouts on rural high-speed roadways are safer than TWSC intersections.

Although this paper accomplished its objective of evaluating the safety of rural high-speed roundabouts, the analysis is limited to only four intersections. Studies based on larger datasets that include more qualified rural high-speed intersection are needed in the future to further testify the safety performance of such roundabouts. Further, a cost-benefit analysis of TWSC intersection conversion to roundabouts is needed that takes into account conversion costs, savings in traffic delays, energy consumption considerations, and costs of accidents.
REFERENCES


<table>
<thead>
<tr>
<th>PAPER TITLE</th>
<th>Vaccines for roads: road assessment program initiatives in India and China</th>
</tr>
</thead>
<tbody>
<tr>
<td>TRACK</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SMITH, Greg</th>
<th>REGIONAL DIRECTOR, ASIA PACIFIC</th>
<th>iRAP</th>
<th>AUSTRALIA</th>
</tr>
</thead>
<tbody>
<tr>
<td>ROGERS, Luke</td>
<td>SENIOR ROAD SAFETY ENGINEER</td>
<td>iRAP</td>
<td>UK</td>
</tr>
<tr>
<td>BHAVSAR, Jigesh</td>
<td>ROAD SAFETY CONSULTANT</td>
<td>iRAP</td>
<td>INDIA</td>
</tr>
<tr>
<td>ZHANG, Tiejun</td>
<td>SENIOR ROAD SAFETY ENGINEER</td>
<td>RESEARCH INSTITUTE OF HIGHWAY</td>
<td>CHINA</td>
</tr>
</tbody>
</table>

| E-MAIL (for correspondence) | greg.smith@irap.org |

KEYWORDS:
Road safety, inspections, star rating

ABSTRACT:
Road Assessments Programs have to date undertaken risk assessments on some 600,000km of roads in 70 countries. This includes 150,000km of roads in low- and middle-income countries, where 9 out 10 road deaths occur and where vulnerable road users typically account for the majority of people killed.

This paper describes key initiatives of two of the largest assessment programs, in India and China. In India, major roads are now being constructed based on designs that have benefited from the application of iRAP Star Rating processes, while ChinaRAP is developing localised tools that are supporting assessments of major roads across numerous provinces and cities and can underpin a long-term national safety program.
Vaccines for roads: road assessment program initiatives in India and China

Greg Smith¹
Luke Rogers¹
Jigesh Bhavsar²
Tiejun Zhang³
¹iRAP
²Consultant, India
³Research Institute of Highway, China
Email for correspondence: greg.smith@irap.org

1 INTRODUCTION

Road Assessment Programs have to date undertaken risk assessments on some 600,000km of roads in 70 countries. This includes 150,000km of roads in low- and middle-income countries, where 9 out 10 road deaths occur and where vulnerable road users typically account for the majority of people killed. iRAP is built on the premise that causes of road trauma are well known, as are ‘vaccines’ to prevent them. There is enormous potential to better invest limited budgets; the United Nations Decade of Action for Road Safety 2011-2020 brings unprecedented international leadership and political-will to the cause.

This paper describes key initiatives of two of the largest assessment programs, in India and China. In India, major roads are now being constructed based on designs that have benefited from the application of iRAP Star Rating processes, while ChinaRAP is developing localised tools that are supporting assessments of major roads across numerous provinces and cities and can underpin a long-term national safety program.

2 ABOUT iRAP ASSESSMENTS

The International Road Assessment Programme (iRAP) is a registered charity dedicated to preventing the more than 3,500 road deaths that occur every day worldwide. iRAP provides tools and training to help automobile associations, governments, road authorities, funding agencies, research institutes and other non-government organisations in more than 70 countries make roads safe. The activities include:

• inspecting high-risk roads and developing Star Ratings, Safer Roads Investment Plans and Risk Maps
• providing training, technology and support that will build and sustain national, regional and local capability
• tracking road safety performance so that funding agencies can assess the benefits of their investments.


3 ASSESSMENTS IN INDIA

By any measure, road crashes represent an enormous public health challenge for India. On many of the nation’s most important roads, school children, factory workers, farmers and people visiting markets vie for limited road space with high-speed trucks, buses and cars. The result is an alarmingly high rate of death and injury nationally, and extremely high rates on particular lengths of road. 137,423 people were reportedly killed in road crashes in India in 2013 (ADSI, 2013), although the World Health Organisation estimates the number could be even higher, totalling 231,027 (WHO, 2013). Today, some 2 million people in India are living with disabilities caused by road traffic injuries (Watkins, 2009). There is strong evidence of a link between road crashes and poverty in India. Nationally, road crashes are reported to cost the economy around INR 3,300 billion (USD 59 billion) per year (WHO, 2013).
This level of road trauma is not an inevitable outcome of rapid development - it is preventable. As part of efforts to curb road deaths and serious injuries, the World Bank Global Road Safety Facility (GRSF) invited the iRAP to work with the Ministry of Road Transport and Highways (MoRTH), public works departments, research institutes, local engineering firms and motoring clubs to assess the safety of Indian roads. Between the first assessments being undertaken in 2010 and 2014, almost 6,500km of roads in seven States – Andhra Pradesh, Assam, Haryana, Kerala, Gujarat, Rajasthan and Karnataka - had been assessed and more than 150 engineers have participated in training. In 2014, new assessments of some 4,000km of roads were initiated, in Tamil Nadu and Uttar Pradesh.

Table 1. World Bank projects making use of iRAP assessments in India

<table>
<thead>
<tr>
<th>Project name</th>
<th>Total value (USD million)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Andhra Pradesh Road Sector Project</td>
<td>645</td>
</tr>
<tr>
<td>Assam State Roads Project</td>
<td>397</td>
</tr>
<tr>
<td>Gujarat State Highway Project II</td>
<td>566</td>
</tr>
<tr>
<td>National Highways Interconnectivity Improvement Project</td>
<td>1,152 *</td>
</tr>
<tr>
<td>Second Karnataka State Highway Improvement</td>
<td>1,003</td>
</tr>
<tr>
<td>Second Kerala State Transport Project</td>
<td>445</td>
</tr>
<tr>
<td>Total</td>
<td>4,208</td>
</tr>
</tbody>
</table>

* Status: pipeline

3.1 The Findings

The inspections use a vehicle equipped with GPS, video cameras, distance measurement devices and survey software, which analysts used to record around 50 different road design attributes that are known to influence the likelihood of a crash and its severity on each 100 metre segment of road. These attributes include intersection design, road cross-section and markings, roadside hazards, footpaths and bicycle lanes. The data collection was led by local firm Indian Road Survey and Management (IRSM) and Public Works Department (PWD) staff.

The inspections found that many of the roads lack the most basic engineering safety features such as footpaths, safety barriers, paved shoulders and safe intersection design. The risk factors analysis summarised in Figure 1 below play a significant role in the risk of death and serious injury and helps to provide a basis for planning life-saving treatments.

Figure 1. Key infrastructure risk factors for roads assessed in India
3.2 Star Ratings

iRAP Star Ratings are based on road inspection data and provide a simple and objective measure of the level of safety which is ‘built-in’ to the road for vehicle occupants, motorcyclists, bicyclists and pedestrians. Five-star roads are the safest while one-star roads are the least safe. Importantly, Star Ratings can be completed independent of detailed crash data, which is often unavailable in low-income and middle-income countries.\(^1\)

Table 2 summarises the Star Ratings for all the roads assessed, and for the roads in each of the seven States. It shows that the majority of roads are rated in the 1- and 2-star bands for all road users. Very few of the roads achieved 4- or 5-star ratings.

<table>
<thead>
<tr>
<th>Star Rating</th>
<th>Vehicle occupants</th>
<th>Motorcyclists</th>
<th>Pedestrians</th>
<th>Bicyclists</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 star</td>
<td>0%</td>
<td>0%</td>
<td>0%</td>
<td>0%</td>
</tr>
<tr>
<td>4 star</td>
<td>2%</td>
<td>1%</td>
<td>0%</td>
<td>0%</td>
</tr>
<tr>
<td>3 star</td>
<td>24%</td>
<td>16%</td>
<td>10%</td>
<td>11%</td>
</tr>
<tr>
<td>2 star</td>
<td>35%</td>
<td>29%</td>
<td>51%</td>
<td>23%</td>
</tr>
<tr>
<td>1 star</td>
<td>37%</td>
<td>53%</td>
<td>36%</td>
<td>51%</td>
</tr>
<tr>
<td>N/A</td>
<td>2%</td>
<td>2%</td>
<td>2%</td>
<td>14%</td>
</tr>
<tr>
<td>Total</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
</tr>
</tbody>
</table>

\(^1\) Andhra Pradesh, Assam, Haryana, Kerala, Gujarat, Rajasthan and Karnataka.

3.3 Supporting Data

Detailed speed measurements and crash investigation studies were also carried out by J P Research so that the true nature of crashes on the roads could be fully understood (JPR 2012a; JPR 2012b). To illustrate the findings, examples from State Highway 20 (SH20) in Karnataka are provided below. SH20 is a two-lane undivided road with paved shoulder of 1.5m width on both sides, divided by white broken line (road marking) all along the stretch.

Speed monitoring showed that 85\(^{th}\) percentile speeds exceed the posted speed limit, which is 60km/h (see Figure 2 below). Cars typically travelled significantly faster than the speed limit, with an 85\(^{th}\) percentile speed of 89km/h. The results also show that there is a large speed differential between vehicle types.

Figure 2. 85\(^{th}\) percentile speeds on sections SH20, Karnataka, with a speed limit of 60km/h

Detailed investigations of 19 crashes were undertaken on SH20 during a 46 day period (from 15 November to 30 December, 2011). In total, 10 fatalities and 22 hospitalizations were recorded. On an annualized basis, this implies a death rate of 0.46 deaths per km per year on the road, which is substantially higher than typical death rates in high-income countries. Figure 3 below shows that most of the fatalities and serious injuries occurred in front-rear and head-on crashes.

\(^1\) Details of the methodology are available at: http://www.irap.org/en/about-irap-3/methodology.
3.4 Safer Roads Investment Plans

Based on the data collected, Safer Roads Investment Plans – which contain the ‘vaccines’ for high-risk roads - were developed. These plans draw on more than 70 proven road safety treatments that include low-cost road markings, pedestrian refuges, intersection upgrades, and full highway duplication. The most comprehensive analysis found that a combined investment for the seven states of INR 60 billion ($1 billion USD) would save almost 20,000 lives and avert almost 200,000 serious injuries equivalent to 163 billion rupees ($2.7 billion USD) in crash costs avoided over the 20 year life of the infrastructure improvements.

Table 3 below summarises key road safety countermeasure options that were identified as being identified economically viable for 104km of roads in Kerala. The table shows, for example, that improving delineation of 169 lane-km of road could prevent 1,342 fatalities and serious injuries over 20 years, generating a benefit cost ratio of 9.2:1. To assist local designers in implementing these types of countermeasures, iRAP produced the Road Safety Toolkit (http://toolkit.irap.org).

<table>
<thead>
<tr>
<th>Countermeasure type</th>
<th>Length (km)</th>
<th>Fatalities and serious injuries prevented</th>
<th>Benefit cost ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Road Surface Upgrade</td>
<td>104</td>
<td>2238</td>
<td>24.8</td>
</tr>
<tr>
<td>Delineation</td>
<td>169</td>
<td>1342</td>
<td>9.2</td>
</tr>
<tr>
<td>Pedestrian Footpath</td>
<td>193</td>
<td>1121</td>
<td>5.3</td>
</tr>
<tr>
<td>Duplication</td>
<td>32</td>
<td>1019</td>
<td>1.9</td>
</tr>
<tr>
<td>Traffic Calming</td>
<td>92</td>
<td>782</td>
<td>5.3</td>
</tr>
<tr>
<td>Pedestrian Crossing</td>
<td>77</td>
<td>679</td>
<td>6.9</td>
</tr>
<tr>
<td>Intersection - Signalise</td>
<td>16</td>
<td>603</td>
<td>3.1</td>
</tr>
</tbody>
</table>

Table 3. Economically viable safety countermeasures for 104km of roads in Kerala (20 year analysis)

3.5 Star Rating of Road Designs

Importantly, investments to improve many of the roads assessed in India have already been committed to, by government and the World Bank. To date, designs for around 25% of the roads assessed have been Star Rated, helping to ensure that safety is built-in to the plans prior to construction. Many of the roads assessed are now being constructed and designs show significant improvements. For example, on about 200km of roads in Kerala the designs now include raised-threshold pedestrian crossings, sidewalks, safety barriers and separate lanes for slow-moving traffic (World Bank, 2013). The length of road rated 1- and 2-stars for vehicle occupants is expected to drop from 76% to just 4%. It is anticipated that the improvements will result in 14,000, or 57%, fewer deaths and serious injuries than would otherwise have occurred. Figure 4 below illustrates how the risk profile for vehicle occupants on one of the roads compares between existing conditions (dark blue) and with the new design (light blue). It shows that overall, the design would

---

have lower risk scores, and therefore better Star Ratings. In fact, the design would have no 1-star sections and just one short segment rated 2-stars.

![Image](image_url)

**Figure 4.** Risk profile for vehicle occupants under existing conditions (dark blue) and design conditions (light blue) on a section of road in Kerala

4 ASSESSMENTS IN CHINA

Like India, China also faces a significant road safety challenge. It is reported that in 70,134 people are killed in road crashes each year, although the WHO estimates that the number of deaths could be 275,983 (WHO, 2013). To help tackle the problem, iRAP and the Research Institute of Highway (RIOH), Ministry of Transport, are collaborating on the development of the China Road Assessment Program (ChinaRAP). ChinaRAP is currently helping to shape development bank projects worth more than USD 2.4 billion (see Table 4 below), as well as numerous government projects.

<table>
<thead>
<tr>
<th>Project name</th>
<th>Total value (USD million)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anhui Intermodal Sustainable Transport Development Project, (ADB)</td>
<td>634</td>
</tr>
<tr>
<td>Yunnan Honghe Prefecture Urban Transport Project (WB)</td>
<td>350</td>
</tr>
<tr>
<td>Jiaozuo Green Transport and Safety Improvement Project (WB)</td>
<td>200</td>
</tr>
<tr>
<td>Shaanxi Mountain Road Safety Demonstration Project (ADB)</td>
<td>373</td>
</tr>
<tr>
<td>Tianjin Urban Transport Improvement Project (WB)</td>
<td>253</td>
</tr>
<tr>
<td>Urumqi Urban Transport Improvement Project</td>
<td>tbd</td>
</tr>
<tr>
<td>Yunnan Pu’er Regional Integrated Road Network Development Project</td>
<td>593</td>
</tr>
</tbody>
</table>

ADB: Asian Development Bank  
WB: World Bank

The assessments in China span both city roads and highways. As an example of a city road project, as part of the Yunnan Honghe Prefecture Urban Transport Project, the ChinaRAP team assessed roads in the cities of Jianshui and Mengzi. Figure 5 below illustrates pedestrian Star Ratings for key roads in Mengzi. It shows that many of the roads are rated in the lower-risk 4- and 5-star categories. This is primarily because there are sidewalks present and speed surveys undertaken with local police (see Figure 6 below) showed that operating speeds largely complied with the 40km/h speed zone for the roads. However, Figure 5 also shows key roads are rated 1- and 2-stars for pedestrians. In the case of the 2-star roads, operating speeds are approximately 60km/h, and on the 1-star road, operating speeds are approximately 100km/h.
A refunctioning of the road zoned 100km/h, to reduced speed limits that are compatible with the urban environment, was identified as being central to road safety initiatives on the Mengzi Road network. Additionally, an analysis identified infrastructure improvements – or ‘vaccines’ - that have potential to reduce risk of death and serious injury in Mengzi by approximately 14%. These include:

- Integration of signalised pedestrian crossings on major roads, particularly at each leg of intersections on major roads.
- Completion of sidewalks and use of pedestrian fences to guide pedestrian to safe crossing locations.
• Additional traffic calming at key high-pedestrian flow locations to ensure traffic speeds remain within safe limits.

• Adjusting and upgrading delineation (lines and signs), particularly at intersections.

• Managing roadside hazards at locations where traffic speeds have the potential to be relatively high.

The Shaanxi Mountain Road Safety Project is an example of an assessment of rural highways. Here, the ChinaRAP team is working with local designers to lift safety star ratings on almost 1,000km. The network comprises single lane undivided roads with narrow or no paved shoulders and few intersections. More than two thirds (71%) of the network is curved and 89% of the roadsides have hazards such as cliffs, steep drains and aggressive vertical faces. The terrain and pavement (51% in medium or poor condition) contribute to relatively low operating speeds, which are often 60km/h or less. Pedestrians and bicyclists tend to only be present in villages and towns, yet facilities are very limited. Overall, the majority of the roads are rated 1- or 2-stars by ChinaRAP. Figure 7 below is typical of the roads.

The key features of preliminary designs that relate to risk of death and serious injury in the project are:

• realignments which reduce the overall length of the road and reduce the number of curves (reduces risk)
• installation of new safety barriers and replacing old safety barriers (reduces risk)
• increasing lane width, adding some paved shoulders and improving pavement condition (reduces risk)
• improving delineation (reduces risk)
• an increase in operating speeds (increases risk).

On average, the preliminary designs are expected to lift vehicle occupant Star Ratings, and in some locations, such as the one shown in Figure 8, the improvement will be marked. Realignments of sections with sharp curves and hazardous roadsides will improve from 1- and 2-stars to 3- and 4-stars. Figure 8 also shows that, with the exception of one short section, the Star Ratings on other sections of road will also increase. This is despite an expected increase in operating speeds. The ChinaRAP team is now working closely with designers to identify further opportunities to lift the Star Ratings for all road users, and therefore reduce risk of death and serious injury.
In addition to undertaking projects in China, the ChinaRAP team is also building valuable international experience. In 2013, the team assisted in the AusRAP assessment of national roads in Australia. In 2014, the team undertook assessments in Yemen with the World Bank that will help shape a number of projects, such as the Second Rural Access Project. The team is now leading road attribute data collection for an innovative KiwiRAP cities project in New Zealand.

The creation of a long-term, sustainable program has been a particular focus for the ChinaRAP team, which now includes 11 staff. Training, and the development of data collection equipment and data management software that can support expansion of risk assessments and integration in provincial and local road authorities, have been key features of the program.

5 CONCLUSIONS

The iRAP programs in India and China are demonstrating the strong potential for a rigorous and data driven road safety assessment and design practices to reduce crash rates and providing models that can be replicated in other provinces within their own countries as well as in other developing countries. A particular strength of the assessments in these countries has been the close link with development bank financed projects, which helps ensure that Star Rating assessments can translate into safer road designs and safer roads. The programs are also effectively building local ownership, which is critical to the long-term sustainability of the work.

6 ACKNOWLEDGEMENTS

iRAP activities in India and China are supported by the World Bank Global Road Safety Facility, Bloomberg Philanthropies, the FIA Foundation and the Road Safety Fund.
7 REFERENCES

Optimizing Crash Cushion Selection based on Performance, Physical Constraints and Reusability

Luigi Grassia1*, Mauro Corsanici2
1Second University of Naples, Department of Industrial and Information Engineering, Via Roma 29, Aversa (CE) 81031, Italy; e-mail: luigi.grassia@unina2.it
2Industry AMS srl, Via Saggese 43, Casalnuovo (NA), Italy; e-mail: m.corsanici@amssrl.com

Abstract: A study able to correlate the crash cushion dimensions to the probability to get an impact is presented. The results show that the probability to get an impact increases as the crash cushion size increases. The probability to get lateral impacts is also included in the study showing that a redirective crash cushion should be desirable in many applications. The efficiency of the crash absorbers mainly affects the length of the crash cushion, whereas its width only depends on the size of the obstacle to protect and does correlate with the efficiency of the absorbers. In particular high efficiency of the absorbers means less length of the and consequently less probability to get an impact. The selection of a crash cushion should be done in order to minimize its life cost: small length and high reusability have a double effect on the life cost because the latter allows to replace only few parts of the crash cushion and the former allows to do the replacement less times than a longer crash cushion.

Introduction
Crash cushions are usually installed to protect obstacles on the roads such as metal cusp of branching roads, tollbooths and at the beginning of central traffic barriers. They are designed to protect the occupants of the vehicle slowing down appropriately the vehicle in case of collision. In Europe the reference standard for the design and dimensioning of crash cushions is UNI EN 1317. The UNI EN 1317 classifies crash cushions depending on the level of speed and the ability to redirect the vehicle in the event of a side impact: there are four speed classes, respectively 50 Km / h, 80 km / h, 100 km / h 110Km/h. Within each speed class a crash cushion may be redirective (in this case it is compatible with side impacts), or non-redirective (in this case it is not able to redirect the vehicle in the event of a side impact). The latter should only be used in places where a side impact on the attenuator is not possible to occur. In Italy the Decree of the Ministry of Infrastructure and Transport of 21/06/2004 provides guidelines for the installation of crash cushions depending on the speed limit of the roads: for roads with a speed limit greater than or equal to 130 km / h it must be installed crash cushions of at least class 100, for roads with speed limit greater than or equal to 90 km / h and less than 130 Km / h it must be installed crash cushions of at least class 80 and finally for roads with speed limit less than 90km / h it must be installed crash cushion of at least 50 class. Products available on the market today are either redirective or non-redirective for each speed class up to 110Km/h. The Italian Decree of 2004 should be revised in the light of advances in technology in recent years, has made available products for the class 110 with costs that are not very different from those of the lower classes. The possibility of installing an attenuator of class 80 on a road with a speed limit of 120Km/h means that in case of a frontal impact at 120 Km/h of a vehicle of 1300 Kg has still, after the attenuator has bottomed out, a residual speed of about 89km/h. The installation in the same point of an impact attenuator of class 110 (tested with vehicles of 1500 Kg at 110km/h) in place of that of class 80 would mean that the residual velocity of the impacting vehicle is only 21Km/h. Therefore the speed class of a crash cushion should be determined in function of the speed limit of the road: the speed class of the impact attenuator should be higher than the speed limit of the
road and in cases where this is not possible, it should be selected the crash cushion class closer to the speed limit of the road

**Life Cycle Cost**

The choice of a crash cushion should be made in function of the life cycle cost (Life Cycle Cost - LCC) rather than to its ex-works price. Indeed, the life cycle cost takes into account all the cost related to the installation and maintenance of the crash cushion over the years. LCC can be calculated with the following equation:

\[ LCC = C_{bw} + C_{pr} \times \left\lfloor \frac{N \times Y}{M} \right\rfloor + C_{re} \times \left( \frac{N \times Y}{M} - \left\lfloor \frac{N \times Y}{M} \right\rfloor \right) \]

where \( C_{bw} \) is the cost of masonry, \( C_{pr} \) is the cost of the product including the installation without the masonry, \( C_{re} \) is the cost necessary to restore the system after an impact if the attenuator is reusable, \( N \) is the number of impacts per year that undergoes the attenuator, \( M \) is the maximum number of impacts that can undergo an attenuator before being fully replaced, \( Y \) is the number of years for which LCC is calculated and finally the function \( \text{int} \) [x] returns the nearest integer to its argument x.

In relation to the costs associated with the restoration (\( C_{re} \)) of an impacted crash cushion, the products available on the market can be classified in three different categories:

1. **Sacrificial**: the crash cushions of this type must be replaced completely after an impact and cannot be restored: This means that \( M = 1 \). Their product cost (\( C_{pr} \)) is generally low. They should be installed in places characterized by a low number of impacts per year (\( N \)) in order to minimize the LCC;

2. **Reusable**: the attenuators of this type are designed to be able to be repaired when impacted by the vehicle in conditions not much different from those of the certification tests; generally the energy absorbing elements must be restored after the impact whereas their support structure can be reused. For them, \( M = 3-4 \), the cost of restoration (\( C_{re} \)) is low (of the order of 20-30% of the cost of the attenuator \( C_{pr} \)) and the cost of the product (\( C_{pr} \)) is intermediate between the sacrificial and low maintenance attenuators. They should be installed in places where the number of impacts per year (\( N \)) is medium in order to minimize the LCC;

3. **Low maintenance**: the attenuators of this type are designed to be able to support a lot of impacts with only minor maintenance. For them, \( M = 5-6 \), restoration costs are very low (of the order of 5-10% of the cost of the attenuator, \( C_{pr} \)) but the cost of the product is generally high. Often the energy absorbing elements are made of hyperelastic material (elastomer or similar) and that case the energy absorbed during the impact is returned back to the car in the post-impact phase. The biomechanical parameters should be checked during the unloading phase in order to ensure the safety of car occupants and avoid effects such as "whiplash". They should be installed in places where the number of impacts per year (\( N \)) is very high in order to minimize LCC.

With reference to equation (1) it is evident that, beyond the trivial reduction of the cost of the product (\( C_{pr} \)), a way to minimize LCC is to increase the number of impacts \( M \) that the attenuator is able to withstand before being replaced and to reduce the number \( N \) of impacts per year that occurs in the place where it is installed. The design of a crash cushion can be oriented both to increase \( M \) and to reduce \( N \). The way of increasing \( M \) is to use a robust approach to the design to ensure that the real behaviour on the roads is not very much different from the behaviour of the crash cushion as evaluated in the certification tests. The number \( N \) of impacts per year in the
place of installation is affected by the size of the crash cushion. The crash cushion, in fact, constitutes itself an obstacle to the motion of the cars on the road and therefore its presence and its dimensions contribute to determine the probability to get an impact in a given position. The width of the crash cushion depends on the size of the obstacle to protect and it is therefore a constraint of the project, the length of the attenuator, instead, is only a function of the efficiency of the energy absorbing elements and of the speed class of the attenuator. Upon fixing the speed class, higher the efficiency of the absorbing elements then shorter and less bulky the crash cushions. Indeed, the problems involved in the development of a crash cushion can be schematized as the problem to stop a certain mass moving at an initial velocity \( V_0 \) in a certain space \( S \). The European normative EN 1317 defines a limit for the maximum deceleration of the vehicle during the impact of a crash cushion through the definition of the ASI parameter.

During the impact the force required to deform the crash cushion affects cushion the level of deceleration of the vehicle, simply because the inertial force \( F_a = m a \) of the vehicle (where \( m \) is the mass of the vehicle and \( a \) the deceleration) equals at each time the force, \( F \), required to deform the crash cushion. Generally crash cushions contain energy absorbers that transform the kinetic energy of the vehicle into internal energy of the absorbers. It is useful to define the efficiency, \( \eta \), of the energy absorbers in terms of force as follows

\[
\eta = \frac{\frac{F_m}{F_{\text{max}}}}{\frac{a_m}{a_{\text{max}}}} = \frac{1}{\int_0^S F(x) dx} \frac{a_m}{a_{\text{max}}} \quad (1)
\]

where \( F(x) \) is the force required to deform the energy absorber, \( x \) is the current deformation of the energy absorber and \( S \) is the maximum deformation that undergoes the absorber, \( F_m \) is the average force and \( F_{\text{max}} \) is maximum force acting on the vehicle. If the absorber works during an impact with a vehicle, it is clear that the efficiency of the absorber equals the ratio of the mean deceleration during the impact and the maximum deceleration during the impact.

Referring to a frontal impact the EN 1317 gives us a limit for \( a_{\text{max}} \): \( a_{\text{max}} < 16.8 \text{ g} = 12g \text{ ASI}_{\text{max}} \), where \( \text{ASI}_{\text{max}} \)=1.4. According to this approach the minimum length of crash cushion can be easily predicted, once the desired ASI and the efficiency of the energy absorber are known:

\[
S = \frac{1}{2} \frac{V_0^2}{\eta \times \text{ASI} \times g} \quad (2)
\]

Equation (2) shows clearly that, independently to the level of velocity \( (V_0 = 50, 80, 100 \text{ e} 110 \text{ Km/h}) \) and fixed the level of the severity of the impact (ASI, in any case < 1.4), the length of the crash cushion (\( S \)) is mainly affected by the efficiency of the crash absorbers (\( \eta \)): for the same level of velocity a shorter crash cushion corresponds to an higher efficiency.
Figure 1. Top view of the junction of the SS 162 NC near Naples (Italy) for which it evaluated the probability to get an impact against the crash cushion; the red attenuator is 5 m long, the blue one is 3m long.

Figure 2. The probability of impact against a 5 m crash cushion over the probability of impacts of 3 m crash cushion as function of the standard deviation of vehicle position.

In order to evaluate how the length of the crash cushion affects the probability to get an impact a quantitative calculation has been carried out at one junction of the SS 162 NC near Naples (Italy) (see Figure 1). It is assumed that the trajectory of the vehicle follows the centre line of line of the
roadway and that the position of the vehicle is a statistical variable whose probability density is described in each instant by a Gaussian distribution with standard deviation equal to 0.3 m. It is then assumed that the volume of traffic on the road in question is equal to 3,000 vehicles per day and that the traveling speed of the same road is equal to 80 Km / h. The probability of impact was then calculated in two different cases: the impact attenuator is 5 m long and the impact attenuator is 3m long. The results of the calculation show that the probability of impact in the case of an attenuator of 5 m is 14 impacts per year, whereas in the case of an attenuator of 3 m the number of impacts per year is reduced up to 8. The calculation was then repeated in a parametric way by varying the standard deviation of the vehicle position and the result is reported in Figure 2. Here the ratio between the number of impacts per year of 5 m attenuator and the number of impacts per year of 3 m attenuator is plotted as function of the standard deviation of the position of the vehicle. From Figure 2 it remains evident that the length reduction of the crash cushion is more effective when the uncertainty on the position of the vehicle is small. In Figures 3a-3d and 4a-4d are reported the results of multibody simulations. The numerical results show that for the same trajectory of the vehicle the installation of an attenuator longer in place of a shorter one of the same class of speed makes more probable the impact of the vehicle and consequently makes more onerous the management of the road.

Figura 3. Multibody numerical simulations of the motion of a car that does impact against a 5 m long crash cushion.
Conclusion
In place where impact attenuators are installed, the probability to get an impact increases as the crash cushion size increases. The calculation of the probability to get lateral impacts is also shows that a redirective crash cushion should be desirable in many applications, whereas non-redirective crash cushions could be dangerous. The efficiency of the crash absorbers mainly affects the length of the crash cushion, whereas its width only depends on the size of the obstacle to protect and does correlate with the efficiency of the absorbers. In particular high efficiency of the absorbers means less length of the and consequently less probability to get an impact simply because the crash cushion is itself an obstacle on the roads. The selection of any crash cushion should be made referring to the LCC rather than to the ex-works price. LCC decreases as the number of impacts per year, N, decreases and N decreases with the length of the crash cushion. It is possible to conclude that, for the same class of speed, the shorter a crash cushion the lower the cost related to its life cycle and the greater the safety of the installation.

Figure 4. Multibody numerical simulations of the motion of a car that does not impact against a 3 m long crash cushion. The trajectory of the car is identical to that reported in figure 3.
Evaluation of the radar detector developed as the next generation automatic incidents detector in Korea

**TRACK**
Integrated Mobility & Intelligent Transportation Systems

<table>
<thead>
<tr>
<th>AUTHOR (Capitalized Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Whoi-bin CHUNG</td>
<td>Senior Researcher</td>
<td>ITS Korea</td>
<td>South Korea</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CO-AUTHOR(S) (Capitalized Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jae-Kyun Lee</td>
<td>Senior Researcher</td>
<td>Metabuild Co., Ltd</td>
<td>South Korea</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CO-AUTHOR(S) (Capitalized Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jin-Ki Lee</td>
<td>Junior Researcher</td>
<td>ITS Korea</td>
<td>South Korea</td>
</tr>
</tbody>
</table>

**KEYWORDS:**
Automatic incident detection
Radar detector
Incident detection
Smart Highway

**ABSTRACT:**
The next generation automatic incidents detector by using radar was developed in smart highway project which was started in 2007 and funded by the ministry of land, infrastructure and transport. The developed radar detector can cover from minimum 500m to maximum 1km. And It aims to detect various events like stationary vehicles, fallen objects, wrong way driving and pedestrian on the expressway.

In this paper, the radar detector was evaluated and analyzed on the test road according to several test scenarios. The test scenarios was composed by obstacle types, speeds(every 10km/h from 60km/h to 140km/h), distance(every 50m), and sizes(30cm, 50cm and 70cm). Each event was evaluated by several performance measurements such as detection rate, detection time, false alarm rate and detecting range.

According to the evaluation results, the developed radar detector has more excellent abilities than the existing automatic incident detectors. Additionally, We found that the radar detector can warn directly the risks of events toward the approaching vehicles through the WAVE(Wireless Access in Vehicular Environments) communication device and also cooperate with vehicle control systems.
Evaluation of the radar detector developed as the next generation automatic incidents detector in Korea

Whoi-bin Chung¹, Jae-Kyun Lee², Jin-Ki Lee¹

¹ITS Korea, Anyang-si, Gyeonggi-do, South Korea
²Metabuild Co., Ltd, Seoul, South Korea

Email for correspondence: jwhoibin@itskorea.kr, jklee@metabuild.co.kr, 0131271@itskorea.kr

1. INTRODUCTION

Traffic experts of Korea believe that traffic incidents such as falling objects, breakdown car and wrong-way driving have threatened drivers on the expressway more seriously than before. In the case of falling objects, 317,000 falling objects became collected on the expressway in 2011. This damage has been identified as significant in scale.

Table 1. Yearly number of fallen objects on the expressway

<table>
<thead>
<tr>
<th>Year</th>
<th>2006</th>
<th>2007</th>
<th>2008</th>
<th>2009</th>
<th>2010</th>
<th>2011</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fallen objects(cases)</td>
<td>261,152</td>
<td>281,815</td>
<td>296,764</td>
<td>307,150</td>
<td>312,829</td>
<td>317,521</td>
</tr>
</tbody>
</table>

According to a report of the Korea Expressway corporation in the end of 2013⁴, for example, 246 people were killed in a expressway because of 408 secondary accidents for the past 5 years (2008–2012). It was recorded that fatality rate is 60 percent, which are 5 times of general traffic accidents (12% fatality rate). These secondary accidents have more caused by falling objects, falling rocks, breakdown car, pedestrian, wrong-way driving and so on, commonly.

Table 2. Analysis result about the fatality of secondary accidents

<table>
<thead>
<tr>
<th>Year</th>
<th>2008</th>
<th>2009</th>
<th>2010</th>
<th>2011</th>
<th>2012</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Accidents(cases)</td>
<td>86</td>
<td>75</td>
<td>94</td>
<td>78</td>
<td>75</td>
<td>408</td>
</tr>
<tr>
<td>Fatality(persons)</td>
<td>41</td>
<td>50</td>
<td>58</td>
<td>47</td>
<td>50</td>
<td>246</td>
</tr>
<tr>
<td>Fatality rate(%)</td>
<td>47.7</td>
<td>66.7</td>
<td>61.7</td>
<td>60.3</td>
<td>66.7</td>
<td>60.3</td>
</tr>
</tbody>
</table>

In the table 2, secondary accidents have relatively broad implication, that is any accident sufficiently proximate to a crash location in time and space. Currently, the Korea expressway corporation does not have the exact numerical criterion about the secondary accidents.

Although the existing various surveillance systems have operated to detect traffic incidents and provide those information to drivers until now, their capabilities, in terms of traffic safety, have reached limits. Thus, road operators are needed the innovative and multipurpose automatic incidents detector overcoming the existing systems' limitations.

MOLIT (Ministry of Land, Infrastructure and Transport) launched Smart Highway project, which is included in VC (Value Creation) 10 projects, in 2007. One of the main objectives is to develop the new-concept detectors for the detection of incidents. Detected incidents information should be directly provided for vehicles on the expressway through WAVE (Wireless Access in Vehicular Environments) which is the well-developed DSRC (Dedicated Short Range Communication) protocol for vehicles.

Developed detectors are Smart-I (eye) using panorama video and Smart-IDS (Incident Detection System) using radar technology. Those have additional capabilities except for incident detection. They can monitor traffic status and surface condition of 1km section on the expressway with 6 lanes and also cooperate with vehicles through WAVE.

A wireless access in vehicular environments (WAVE) system is a radio communication system intended to provide seamless, interoperable services to transportation. These services include those recognized by the U. S. National Intelligent Transportation Systems (ITS) architecture and many others contemplated by the automotive and transportation infrastructure industries around the world, such as communications between vehicles and infrastructure, and communications among vehicles.⁵

This paper presents about Smart-IDS and its evaluation results.
2. Smart-IDS CONFIGURATION

2.1 SYSTEM COMPONENTS

The automatic incident detector using radar detector is composed of radar detector, local server and system controller (see figure 1). The role of each sub-system is as below.

- Radar detector: Detection of abnormal conditions on the roadway by analyzing the received signal
- Local server: Generation and transmission of detection data, Control of radar detectors and tracking camera
- System controller: Management of local servers and radar detectors through software based on GUI(Graphic User Interface).

![Smart-IDS configuration](image)

**Figure 1. Smart-IDS configuration**

The radar detector is largely composed of antenna, transceiver, and signal processor module (see figure 2). Antenna is an electrical device which converts electric power into radio waves, and vice versa. Transceiver is a device comprising both a transmitter and a receiver which converts up/down frequency between antenna and signal processor. Lastly, Signal processor performs signal processing algorithm (pulse compression, pulse integration, Doppler processor, CFAR etc.) using transmitted signal from transceiver.

![Block diagram of a radar detector](image)

**Figure 2. Block diagram of a radar detector**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Specification</th>
<th>Parameter</th>
<th>Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frequency</td>
<td>34.5GHz(Ka-band)</td>
<td>Sampling rate</td>
<td>200MHz</td>
</tr>
<tr>
<td>Peak Power</td>
<td>100mW</td>
<td>Pulse Width</td>
<td>6.67ns</td>
</tr>
<tr>
<td>PRF</td>
<td>30KHz</td>
<td>Range resolution</td>
<td>1m 이상</td>
</tr>
<tr>
<td>Bandwidth</td>
<td>150MHz</td>
<td>Velocity resolution</td>
<td>1.6km/h(0.46m/s)</td>
</tr>
<tr>
<td>Type</td>
<td>Pulse Doppler</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table 3. Specification of a radar detector**

![Prototype pictures of a radar detector](image)

**Table 4. Prototype pictures of a radar detector**
2.2 FEATURES
The automatic incident detector system feature as follow:
- Road Monitoring
  - Detect fallen object
  - Detect stopped/reversed vehicle
  - Detect vehicle speed, position
- Event Alarm
  - Incident event alarm
  - Tracking Camera linking to system
  - Integrate other system
- Provide Information
  - Additional traffic information
  - Create and provide event information
  - Provide info to other system by trigger

2.3 PERFORMANCE
- Detectable Area
  - Number of lanes: up to 5
  - Detection range: 30m to 1000m
  - Any lane spacing is supported
- Performance
  - Detection Accuracy: up to 95%
  - Detection Speed: -200kph ~ 200kph
  - Range resolution: up to 1m
  - Velocity resolution: ±1kph

2.4 APPLICATION
- Verification system of falling objects/broken cars on the road
- Verification system of road surface status
- Verification system of wild animals and pedestrians
- Road traffic measurement and speed enforcement system

3. PERFORMANCE EVALUATION METHOD

3.1 PERFORMANCE EVALUATION ITEMS
To define the criterion, several things were considered. First, is there any reference criterion? Second, what is the researcher’s objective? According to the results of comparison of any reference criterion and the researcher’s objective, the researcher’s goals were higher and so was selected as the final criterion. This criterion is as follows.

<table>
<thead>
<tr>
<th>Performance evaluation item</th>
<th>Reference criteria($)</th>
<th>Researchers' goals</th>
<th>Target criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Detection rate</td>
<td>More than 80%</td>
<td>More than 90%</td>
<td>More than 90%</td>
</tr>
<tr>
<td>Mean time to detection</td>
<td>Less than 5 minutes</td>
<td>Less than 1 minute</td>
<td>Less than 1 minute</td>
</tr>
<tr>
<td>False alarm rate</td>
<td>Less than 1%</td>
<td>Less than 1%</td>
<td>Less than 1%</td>
</tr>
<tr>
<td>Maximum detection range</td>
<td>N/A</td>
<td>Within 500 meters</td>
<td>Within 500 meters</td>
</tr>
</tbody>
</table>

3.2 PERFORMANCE EVALUATION CASES
Generally, incident types can be classified as stopped vehicles, obstacle, non-recurring congestion and wrong way driving. It is impossible to generate all kinds of incident types in real. Thus, we defined below test cases in detail unlike general incident types.

- Stopped vehicle: The case is to stop a vehicle by 10 km/h intervals between 40km/h and 80km/h, and by each 50m and lanes. The results were validated whether the events were detected within 60 seconds.
- Obstacle on the road: The case is to fall a box by every 50 meters and by every lanes. The box is thrown in the car.
- Normal driving: The case is to check the false alarms while driving a car by every 20km/h intervals between 20km/h and 140km/h.
- Wrong way driving: The case is to check a wrong-way driving car by every 20km/h intervals between 20km/h and 140km/h.

And, before starting official test, the operation status of smart-IDS is checked by using a standard cylinder. In real, testers verified lane separation ability and distance precision related to the detection position notification.

### Table 6. Representative pictures by test cases

<table>
<thead>
<tr>
<th>Stopped vehicle</th>
<th>Fallen object</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1" alt="Stopped vehicle" /></td>
<td><img src="image2" alt="Fallen object" /></td>
</tr>
<tr>
<td>Normal driving and wrong wary driving</td>
<td>Appearance of people</td>
</tr>
<tr>
<td><img src="image3" alt="Normal driving" /></td>
<td><img src="image4" alt="Appearance of people" /></td>
</tr>
</tbody>
</table>

### 3.3 SEPERATION OF RANGE FOR EVALUATION

In case of a detection test about stopped vehicle, a test car will stop in random place every 50m and all lanes, 2 lanes and a shoulder lane. Objects in the shoulder lane will apparently have some problems so we think that moving objects as well as stopped objects have to be detected necessarily. In another case of obstacles on the road, we randomly dropped the obstacles on the road at 50-meter interval in a monitoring area.

### Table 7 The stopping or dropping position in the monitoring area

<table>
<thead>
<tr>
<th>Distance from a detector</th>
<th>50m</th>
<th>100m</th>
<th>150m</th>
<th>200m</th>
<th>250m</th>
<th>300m</th>
<th>350m</th>
<th>400m</th>
<th>450m</th>
<th>500m</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st lane</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2nd lane</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shoulder lane</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### 3.4 TEST(EVALUATION) ENVIRONMENTS

An obstacle was made by plastics and thick papers. The colors are black and blue and the sizes of an obstacle are 50cm(H)*50cm(D)*50cm(W) and 30cm(H)*30cm(D)*30cm(W).
4. EVALUATION RESULTS

Looking at the table 8, most of evaluation values exceeded the target value on average. In the event of a stopped vehicle and a wrong way driving, detection rate is 100% perfectly. The results of detecting 30cm and 50cm obstacles shows 90% and 87.7% detection rate, respectively. The size of an obstacle did not affect on the detection ability in this test. The detection time was 7.98 seconds on average. False alarm rate of 0% occurred, which is below the target value.

<table>
<thead>
<tr>
<th>Performance measures</th>
<th>Test scenario</th>
<th>Target values</th>
<th>Test result</th>
</tr>
</thead>
<tbody>
<tr>
<td>Detection rate</td>
<td>Stopped vehicle</td>
<td>More than 90%</td>
<td>100%</td>
</tr>
<tr>
<td></td>
<td>Fallen object(30cm)</td>
<td></td>
<td>87.7%</td>
</tr>
<tr>
<td></td>
<td>Fallen object(50cm)</td>
<td></td>
<td>90%</td>
</tr>
<tr>
<td></td>
<td>Wrong way driving</td>
<td></td>
<td>100%</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td></td>
<td>94.4%</td>
</tr>
<tr>
<td>Mean time to detection</td>
<td>Stopped vehicle</td>
<td>Less than 1 minutes</td>
<td>8.53 sec</td>
</tr>
<tr>
<td></td>
<td>Fallen object(30cm)</td>
<td></td>
<td>8.27 sec</td>
</tr>
<tr>
<td></td>
<td>Fallen object(50cm)</td>
<td></td>
<td>7.14 sec</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td></td>
<td>7.98 sec</td>
</tr>
<tr>
<td>False alarm rate</td>
<td>Total</td>
<td>Less than 1%</td>
<td>0%</td>
</tr>
</tbody>
</table>
5. CONCLUSION AND FUTURE PLAN

According to the evaluation results, the developed radar detector has more excellent abilities than the existing automatic incident detectors. Additionally, We found that the radar detector can warn directly the risks of events toward the approaching vehicles through the WAVE(Wireless Access in Vehicular Environments) communication device and also cooperate with vehicle control systems.

Evaluation results were satisfied our proposed criterion. But, this research still has limitations because the test road was not a real traffic situation. Thus an additional validation and verification are needed. Currently, Korea Expressway corporation is operating and testing at two sites on the expressway.

Table 9. Two sites operation status on the expressway

<table>
<thead>
<tr>
<th>Location</th>
<th>Gyeong-bu expressway</th>
<th>Seohae Grand Bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>Installation</td>
<td><img src="image1.png" alt="Image" /></td>
<td><img src="image2.png" alt="Image" /></td>
</tr>
<tr>
<td>Sample image</td>
<td><img src="image3.png" alt="Image" /></td>
<td><img src="image4.png" alt="Image" /></td>
</tr>
</tbody>
</table>

7. ACKNOWLEDGEMENT

This research was supported by a grant from Construction Technology Innovation Program(CTIP) funded by Ministry of Land, Infrastructure and Transport (MoLIT) of Korean government.
8. REFERENCES

[1] Jae-hyoung Park, In-chul Jung, Seo-yeong Jang, Min-hong Han, A Study on Road Control System in Smart Highway, The Korean Institute of Communications and Information Sciences, 2008 Summer Conference, July 2008


[7] Thomas Wells, Eric Tofflin, VIDEO-BASED AUTOMATIC INCIDENT DETECTION ON SAN-MATEO BRIDGE IN THE SAN FRANCISCO BAY AREA, 12th World Congress on ITS, 6-10 November 2005

[8] Dr. Peter T. Martin, Associate Professor, Joseph Perrin, Ph.D., PE, PTOE, Blake Hansen, M.S., “INCIDENT DETECTION ALGORITHM EVALUATION”, University of Utah, March 2001


[12] Yunlong Zhang, Lori M. Bruce, AUTOMATED ACCIDENT DETECTION AT INTERSECTIONS, Mississippi Transportation Research Center, March 2004
PAPER TITLE: ROAD PROJECT DELIVERY METHOD SELECTION MODEL: A REVIEW FOR INDONESIAN ROAD DEVELOPMENT

<table>
<thead>
<tr>
<th>TRACK AUTHOR</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nazib Faizal</td>
<td>Researcher</td>
<td>Institute of Road Engineering, Agency for R&amp;D, Ministry of Public Works</td>
<td>Indonesia</td>
</tr>
<tr>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

EMAIL: nazib.faizal@pusjutan.pu.go.id

KEYWORDS: project delivery system, selection model, design, bid, build, maintenance

ABSTRACT:

Road Project Delivery Method Selection Model has been developed by researcher to address evolution of traditional method (Design-Bid-Build). Various tools to select the model base on several parameters. This paper reviews the existing selection model that has been used by road authority in several countries for reference in Indonesia selection model. There are 4 phases for selecting road project delivery system in Indonesia base on existing condition includes: project definition and goal, site investigation and landscaping, selection, and judgment experts. Several factors can affect selection phase.
Road Project Delivery Method Selection Model: A Review for Indonesian Road Development

Nazib Faizal

1Institute of Road Engineering, Bandung, Jawa Barat, Indonesia

Email for correspondence: nazib.faizal@pusjatan.pu.go.id

1. INTRODUCTION
Indonesia is pursuing a goal of Acceleration and Expansion of Indonesia Economic Development 2011-2025 Master Plan (MP3EI) that provides the building blocks to transform Indonesia into one of the 10 major economies in the world by 2025. To achieve this, real economic growth must reach 7-9 percent per year, on an on going basis (CMEA, 2011).
CMEA (2011) stated that the implementation strategy of MP3EI will integrate 3 main elements: 1) Developing the regional economic potential in 6 Indonesia Economic Corridors, 2) Strengthening national connectivity locally and internationally, and 3) Strengthening human resources capacity and national science and technology to support the development of main programs in every economic corridor.
Road development is one of program to strength national connectivity. It takes many efforts such as funding, technology, human resources, organization, and type of project delivery systems.
Anyhow, Directorate General of Highways (DGH) that conduct national road development in Indonesia still use traditional delivery systems which include 3 main phases: 1) design phase, 2) bid phase, and 3) build phase. Another delivery system such as Design-Build, PMC, and etc. are not commonly use in Indonesia. It just started in 2011 when DGH implemented Design-Build Delivery System with Performance Based Contract scheme. DGH needs various delivery systems that can be used in road development. The big question is how to select road project delivery method which proper with Indonesian condition?

2. INDONESIA ROAD DEVELOPMENT AND DELIVERY SYSTEM
Base on Government Regulation No. 34 Year 2006 on Road, Road development is activity that includes programming and budgeting, technical design, construction, operation, and maintenance (Figure 1).
Figure 1 Indonesia Road Development base on Government Regulation No.34/2006

DGH performs programming and budgeting while the contract winner (consultant and contractor) conduct technical design and construction. Technical design and construction are mostly delivered by Design-Bid-Build in National Road Development (Soemardi and Pribadi, 2010; Rahadian, 2009; Dewi, Too, and Trigunarsyah all, 2012). Operation and maintenance can deliver by in house delivery system or contracted out to consultant and contractor. DGH started design and build (D-B) delivery system in 2011 with Performance Base Contract (PBC) program. PBC tends to increase contractor innovation to manage the road. It happened in 2011 when Bojonegoro-Padangan link when contractor innovates the new system in facing geotechnical problems (DGH, 2013).

Implementing alternative project delivery systems in Indonesia is not simple works. Lack of regulation and legal framework; and lack of experience, skill and knowledge are the reasons why DB project delivery system is not widely implemented (Dewi et.al, 2012).

3. ROAD PROJECT DELIVERY SYSTEM AND SELECTION MODEL

3.1 World Practice

In the post-World War II era, the delivery mechanisms for road rely upon a single system, design-bid-build (DBB) (Miller, 1995). A little over 35 years ago, the client of the construction industry had only limited choice of project delivery system method and many alternatives of project delivery system that the client can choose since 12 catalyst involved (Assworth, Hogg, and Higs, 2013): 1) government intervention, 2) pressure groups being formed to crate change for the benefit, 3) international comparisons, 4) the apparent failure of the construction industry and its associated professions to satisfy the perceived needs of its customers in the way that the work is organized, 5) the increase in PFI projects and planned expansion of PFI policy, 6) the influence of developments in education and training, 7) the impact from research studies into contracting methods, 8) the response from industry, especially in times of recession, towards greater efficiency, and profitability, 9) changes in technology, particularly information technology, 10) the attitudes towards change and the improved procedures from the professions, 11) the clients desire for single point responsibility, and 12) the publication of headliner reports: in 1994, the Latham Report, and in 1998, the Egan report. Social and economic stability are the incentives for seeking alternative road project delivery system (Pakalla, 2002; Koppinen and Lahdeperä, 2004)

A number of studies suggest that alternative project delivery system can add value by cost and time reduction. With 450,000 KM road length in Indonesia and pursuing road serviceability, DGH needs various project delivery system with value for money approach. For instance: Australia saves 16% cost, increase 16% productivity, and increase 22% of quality when use alternative project delivery system (Segal, Moore, and McCarthy, 2003). Other study said that design-build is faster than design-bid-build with same cost relatively (Shresta, O. Connor, and Gibson, 2012)

There are 3 general road project delivery system: 1) Design-Bid-Build (DBB), 2) Design-Build (DB), and 3) Construction Manager/General Contractor (CM/GC) (Rashid, Taib, Basiron, Nasid,
Ali, Nordiana, and Zainordin, 2006; Tran and Molenaar, 2012). Pakalla (2002) divides into two project delivery systems: capital projects and maintenance contract. Capital projects refers to the infrastructure that is built is not to be maintained such as road widening or new roads. While maintenance contract refers to the maintenance of roads such as patching, overlay, and others. Capital projects generally use traditional procurement models (design-bid-build), design-build, design-build-operate-maintain, design-build-finance-operate, and program management. Other innovative models are partnering and lane rentals. As for the maintenance contract consists of several traditional procurement such as the duration of 3-5 years, a hybrid type, Longer-term maintenance contracts, and Performance Specified Maintenance Contracts (PSMC - Outcome-Based Criteria). Assworth et.al (2013) gives 4 project delivery system options: 1) traditional, 2) design and build, 3) construction management, and 4) design and manage. Koppinen and Pertti (2007) divides project delivery system into 4 type: 1) design–bid–build, 2) construction management, 3) design–build, and 4) design–build–maintain.

<table>
<thead>
<tr>
<th>No</th>
<th>Project Delivery System</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Design-Bid-Build (DBB)</td>
<td>Miller, 1995</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Pakalla, 2002</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Rashid et.al, 2006</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Koppinen and Pertti, 2007</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Tran and Molenaar, 2012</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Shresta et.al, 2012</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Assworth et.al, 2013</td>
</tr>
<tr>
<td>2</td>
<td>Design-Build (DB)</td>
<td>Pakalla, 2002</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Rashid et.al, 2006</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Koppinen and Pertti, 2007</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Tran and Molenaar, 2012</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Shresta et.al, 2012</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Assworth et.al, 2013</td>
</tr>
<tr>
<td>3</td>
<td>Construction Management/General Contractor (CM/GC)</td>
<td>Rashid et.al, 2006</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Koppinen and Pertti, 2007</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Tran and Molenaar, 2012</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Assworth et.al, 2013</td>
</tr>
<tr>
<td>4</td>
<td>Design-Build-Maintain (Manage)</td>
<td>Pakalla, 2002</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Koppinen and Pertti, 2012</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Assworth et.al, 2013</td>
</tr>
</tbody>
</table>

*Government provides funding for the project delivery system*

Road Project Delivery System Selection Method above depends on several things that connect to element of project itself. Factors affect project delivery system selection has been widely discussed by researchers. Tran et al (2013) divided the factors that influence the selection of procurement models to 8: 1) delivery schedule, 2) complexity and innovation, 3) level of design, 4) initial project risk assessment, 5) cost, 6) staff experience/availability, 7) level of oversight and control, and 8) competition and contractor experience. Contract mechanism, risk profile,
packaging, and market environment are considered factor for project delivery system selection model (Department of Main Roads, 2006).

Decision-making of Project Delivery System is key for success in determining the road project. Responsibility, the owner’s willingness to be involved, the owner’s in-house technical capability, risk allocation, and the owner’s willingness to control over design are found to be the most key factors in PDS decision-making (Liu, Hio, Shen, Yang, Meng, and Xue, 2014). The development of structured procedure and decision support system will give benefit to owner for selecting project delivery system. Anderson and Oyetunju (2006) allocate 12 options for owner to select the most appropriate project delivery and contract delivery by SMART (simple multi-attribute rating technique with swing weights).

Since construction project is a complex with many uncertain and undefined parameter, decision-making process to select appropriate project delivery system is more complicated. Mafakheri, Dai, Slepek, and Nasiri (2007) used a decision aid model using the analytical hierarchy process (AHP) included 3 parts: 1) the hierarchy structure, 2) the pairwise comparisons matrix, and 3) the method for calculating priorities. Schedule, quality, complexity, value engineering, external approval, risk, uniqueness, culture, cost, financial guarantee, experience, scope change, and size of project are criteria for selecting the optimal project delivery system. Grey Analysis with 3 weighting profiles can compare method of project delivery system (Lo, 2009). Thus, owner can choose the system base on their goal such as minimize the time and cost, cost performance, and value generation and potential.

3.2 Indonesian practice

DGH has important role for road development in Indonesia. They have 11 regional office spread from Aceh Province to Papua Province and conduct 3 main program for road development: 1) maintenance, enhancement, and 3) new road construction. DGH conducts 3 programs for bridge development: 1) maintenance, 2) replacement, and 3) new bridge construction/duplication. Table 2 describes all activities of the programs with method of project delivery.

Project Delivery System in DGH uses 3 method: 1) in house, 2) design-bid-build, and design-build-maintenance. All method still base on complexity of the project and sharing the risk to contractor or consultant. There are no decision-making tools to select the appropriate project delivery system that match with the project. All decisions do not come from structured method, just base on DBB practices that do not meet expectation.

Since Indonesia divided in 3 regions: 1) developed area, 2) underdeveloped area, and 3) developing area, location of the projects must be considered. Technical skill and experiences of road development stakeholders are different. The parameters should be included in selection model.

Small (routine) maintenance usually uses in house method with fix budget (example: US $ 5,000/km). Different from United States, and other countries in house method is used for enhance capacity of young engineer. The Department pushes them to learn how to design, how to construct, how to supervise, and how to maintain the road with small scale. Indonesia may adopt the purpose of in-house method.
Most of the project uses Design-Bid-Build method except for small maintenance. Commonly, one project will be finished in 2 year; 1 year for design including the bidding process and 1 year for construction including the bidding process. When construction is finished, contractor has obligation for warranty, usually for 2 years and some projects use 4 years with Extended Warranty Period Scheme. There are still situation for implementing Design-Bid-Build method: 1) consuming time and 2) quality. Rashid et.al (2006) stated that Design-Bid-Build is the best method for taking high quality. Unlike in Indonesia, there are some issues for quality even using Design-Bid-Build.

DGH starts Design-Build-Maintenance method in 2011 with Performance Base Contract (PBC) scheme. Six projects used this method and there was improvement especially for contractor innovation since government authorizes design-build method for the projects and multi years contract. Nevertheless, lack of regulation and legal framework; and lack of experience, skill and knowledge are the reasons why DB project delivery system is not widely implemented (Dewi et.al, 2012). Moreover, political will must be considered to secure budget especially for multiyear contract that used for PBC scheme. Sultana, Rahman, and Chowdbury (2012) noted political influence and corruption are the hardest obstacles for any new concepts to be implemented in developing countries.

Table 2 Project delivery system, program, and activities

<table>
<thead>
<tr>
<th>NO</th>
<th>Method</th>
<th>Programs</th>
<th>Activities</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>In-house</td>
<td>Routine Maintenance-Road (A) dan Bridge(B)</td>
<td>A: Patching, crack filling, grass cutting, minor maintenance for pedestrian, kerb painting, road cleaning, dan sloping, drainage cleaning, station/bench mark enhancement, and etc.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>B: Bridge cleaning, small painting, and etc.</td>
</tr>
<tr>
<td>2</td>
<td>Design-Bid-Build</td>
<td>Periodic Maintenance-Road (A) and bridge (B), structure improvement (C) dan enhancement road capacity(D), replacement-bridge (E), new road construction (F), and new bridge construction.</td>
<td>A: Overlay and etc.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>B: Overlay, Painting, and etc.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>C: Structural overlay and etc.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>D: Road widening and etc.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>E: Bridge replacement</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>F: New road construction</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>G: New bridge construction/Duplication</td>
</tr>
<tr>
<td>3</td>
<td>Design-Build-Maintenance with PBC scheme.</td>
<td>All programs in point 1 and 2.</td>
<td>All activities in point 1 and 2.</td>
</tr>
</tbody>
</table>
4. DISCUSSION

4.1 Improvement of Existing Project Delivery System

Existing project delivery system needs to be improved before defining project delivery system selection model. There are still some issues regarding to DBB practices: 1) obscurity in term of reference, 2) quantity issues, 3) site investigation issues, 4) output of each process is not well defined, and 5) quality issues. Improvement is base on effectiveness and output clearness on each process during phase of DBB. Similar with DBB, in-house practice is still facing several issues such as: 1) paradigm that in-house system is just for maintenance and 2) quality of maintenance with in-house system. Design-Build-Maintain data is very limited, only 6 contracts from 2011 until 2013 and at least 3 points need to be evaluated: 1) project selection for PBC, 2) lack of human resources, 3) lack of knowledge, 4) lack experiences, 5) lack of data, 6) length of road for PBC is too short, and 3) planning (Murjanto, 2014).

Propose of New DBB Model is adding site investigation (including: survey and mapping, traffic survey, and geotechnical investigation) and architectural before design process. Document of site investigation is a part of design document bid. Company who win the bid only responsible for design because all document requirements is already there. To make construction document bid, the new model uses certified quantity surveyor.

In house and PBMC can deliver maintenance project. Road ranger can improve quality of maintenance since the ranger responsible as “an eye” on the road and supports data and information as decision tools for road manager.

Design-Bid-Maintain (DBM) can be improved by 1) capacity building of human resources and 2) legal framework of DBM. Some stakeholders are still thinking that DBM with PBC scheme is better than DBB while conceptually DBB with PBC scheme is another option to deliver road development. No PDS is better than other, it is just only how to put right PDS on the right project and right place. Dewi et.all (2012) suggests integration PDS can be improved by clear regulation, enlightening project appropriate for integrated project delivery system, project procedure and design builder entity and enhancing communication, training and knowledge sharing.

5. Selection model

Selection model of appropriate PDS in Indonesia can use several factors such as cost, schedule, quality, complexity, scope change, experience, value engineering, financial guarantee, and risk and culture (Mafakheri et.all, 2007) combine with level of road data information availability, legal framework, and goal of the project (Faizal, 2014). Cost is very important for all projects and one of owner’s concerns. Indonesia is trying to achieve MP3EI goal in 2025, thus government push road development in shortest time. We can still find bridge collapses, poor road condition in Indonesia which quality doesn’t meet requirements generally. Selection model must consider complexity of the project (standard of the project, repetitive, or complex/unique design). Scope change or variation order that may occur in construction phase makes owner to reflect which proper PDS for his project. Experience is related to human resources capacity. Owner, consultant, and contractor
experiences bring over the success of selected PDS. Value engineering can deliver innovation. Integrated PDS has more chance to use value engineering especially PDS with performance-based schemes. Indonesia doesn’t have provision or article that manage value engineering in DBB PDS. Consequently, contractor doesn’t have intention to conduct value engineering during construction. Integrated PDS makes contractor has more financial risk. It constrains that contractor has financial guarantee to run the projects. Each process in PDS faces the same risk. The big question is who will get more risk than other. In this case, risk management is very important. Contractor will take the higher risk when use integrated PDS.

Indonesian Road Project Delivery System selection model is not yet structured, the selection is just based on trial and culture. There is paradigm that in house system is the best for maintenance or DBB.

New Road Project Delivery System in Indonesia includes 5 modes: 1) in-house, 2) Design-Bid-Build, 3) Design-Build, 4) Design-Build-Maintain (PBC scheme), and 5) PBMC (PBC for Maintenance Scheme). These 5 PDS will be selected in 4 phases: 1) Project definition and goal, 2) Site Investigation and Landscaping, 3) Selection, 4) Expert Judgment. Project definition and goal in Indonesia is very important. Many failures of delivery caused by unclear statement of project definition and goal. Site investigation and landscaping usually conduct in design phase. In the new model, it separated from design process. Similar to project definition, poor site investigation and landscaping can convey to unsuccessful project delivery. Selection model will use Multi-Criteria Decision Analysis (MCDA) with the factors above. The expert will judge the final PDS base on previous process. Indonesia still needs 4th phases because new PDS except in-house and DBB is still in developing process.

6. CONCLUSION

The purpose of this paper is to examine existing road project delivery system in Indonesia and its selection model. Indonesia has 3 projects delivery system for road development: 1) in-house, 2) design-bid-build, 3) design-build-maintenance with Performance Base Contract scheme. In-house method is only used for small/routine maintenance while design-bid-build is used for periodic maintenance, structural improvement, and new construction. Most of the project use Design-Bid-Build. Design-Bid-Maintenance with PBC scheme has started from 2011 and still being developed by DGH.

Selection model of appropriate PDS in Indonesia can use several factors such as cost, schedule, quality, complexity, scope change, experience, value engineering, financial guarantee, and risk and) combine with level of road data information availability, legal framework, and goal of the project.

New Road Project Delivery System in Indonesia includes 5 modes: 1) in-house, 2) Design-Bid-Build, 3) Design-Build, 4) Design-Build-Maintain (PBC scheme), and 5) PBMC (PBC for Maintenance Scheme). These 5 PDS will be selected in 4 phases: 1) Project definition and goal, 2) Site Investigation and Landscaping, 3) Selection, 4) Expert Judgment.
7. ACKNOWLEDGEMENT

The writer gratefully acknowledge Institute of Road Engineering, Agency of Research and Development, Ministry of Public Work for providing funding to support this research study.

8. REFERENCES

A. P Dewi, E. Too, and B Trigunarsyah (2012). “Implementing Design Build Project Delivery System in Indonesia Road Infrastructure Projects”.


B. Soemardi and K. S. Pribadi (2010). “The Role Of Central Local Agencies in Indonesia's Road Project Delivery System”.


<table>
<thead>
<tr>
<th>PAPER TITLE (90 Characters Max)</th>
<th>The mechanical and thermal analysis of porous asphalt concrete containing steel slags</th>
</tr>
</thead>
<tbody>
<tr>
<td>TRACK</td>
<td>Academia</td>
</tr>
<tr>
<td>AUTHOR (Capitalized Family Name)</td>
<td>POSITION</td>
</tr>
<tr>
<td>Yu-Min SU</td>
<td>Assistant Professor</td>
</tr>
<tr>
<td>CO-AUTHOR(S) (Capitalized Family Name)</td>
<td>POSITION</td>
</tr>
<tr>
<td>Dana MUTIARA</td>
<td>Graduate Research Assistant</td>
</tr>
<tr>
<td>CO-AUTHOR(S) (Capitalized Family Name)</td>
<td>POSITION</td>
</tr>
<tr>
<td>Jyh-Dong LIN</td>
<td>Professor</td>
</tr>
<tr>
<td>E-MAIL (for correspondence)</td>
<td><a href="mailto:yuminsu@kuas.edu.tw">yuminsu@kuas.edu.tw</a></td>
</tr>
</tbody>
</table>

KEYWORDS: Porous Asphalt Concrete, Steel Slag, Thermal Analysis, Skid Resistance, Resilient Modulus
ABSTRACT:

The steel slag is a by-product in the cooling process of steel making when the separation of impurities from the molten steel liquid takes place. The objective of this study was to assess two steel slags, namely Basic Oxygen Furnace (BOF) and Baoshan Slag Short Flow (BSSF), to be added in the Porous Asphalt Concrete. The fundamental consensus of aggregates and steel slag used were investigated. 30% and 50% of steel slags were replaced to design and produce specimens to test in the laboratory. Performance tests including resilient modulus, skid resistance, and thermal analysis were assessed. The PAC mixture containing 30% of BOF slag exhibited a rather better performance with the higher resilient modulus at 10°C, better skid resistance, and highest thermal conductivity in comparison with that with 50% BOF and 30% BSSF. The results from this study indicates that increasing the percentage of aggregate substitution may increase the performance of PAC, however the performance will decrease at the certain percentage.

The mechanical and thermal analysis of porous asphalt concrete containing steel slags

Yu-Min SU¹, Dana MUTIARA², Jyh-Dong LIN³

¹ Assistant Professor, Department of Civil Engineering, National Kaohsiung University of Applied Sciences, 415 Chien-Kung Rd., Sanmin, Kaohsiung 80778, Taiwan
² Graduate Research Assistant, Department of Civil Engineering, National Central University, 300 Chung-Ta Rd., Chungli, Taoyuan 32001, Taiwan
³ Professor, Department of Civil Engineering, National Central University, 300 Chung-Ta Rd., Chungli, Taoyuan 32001, Taiwan

Email for correspondence: yuminsu@kuas.edu.tw

1. INTRODUCTION

The iron and steel industry produces extremely large amounts of slag as byproduct of the iron making and steelmaking processes, and is therefore continuing to develop slag reduction and recycling technologies and intermediate treatment technologies. Slag is a major by-product of the steelmaking procedure; its sources usually comes from the extraction of iron ore to pig iron in a blast furnace (BF) or the conversion of pig iron to raw steel in a basic oxygen furnace (BOF). Based on Executive Yuan of the Environmental Protection Administration, current annual production of slags are approximately 4.6 million tons which contain 65% of BF slag, 24% of BOF slag, and 11% of other slag. It was already discovered that steel slag can be used as aggregates in road pavements. The slag is usually added as part of the coarse aggregate fraction of the mixture at a percentage of 20% to 100%, depending on the application of the mixture.

Steel slag can be used not only in the dense mix asphalt but also can be used in porous asphalt. Porous asphalt pavement is one alternative solution to the problem of storm water drainage from parking and other low traffic density areas. In operation, this type of pavement allows incipient rainfall and local runoff to soak through the pavement surface course of open graded asphalt concrete mix and accumulate in a porous base consisting of large open graded gravel from which the water
would percolate into the natural ground below, if this is possible, or would drain laterally to a sump or channel.

The overall benefits of porous asphalt pavements may include both environmental and safety benefits including improved storm water management, improved skid resistance, reduction of spray to drivers and pedestrians, reduction in light reflection and headlight glare, as well as a potential for noise reduction. There are, however, some disadvantages of this pavement type. In general there is a lack of technical expertise in such types of pavements particularly in cold weather, clogging potential, potential risk of groundwater contamination, potential for toxic chemicals to leak into the system, and potential for anaerobic conditions to develop in underlying soils if unable to dry out between storm events. Porous asphalt pavement has shorter service life than that of dense mix layers.

From the previous study already prove that using steel slag as aggregate can improve the performance of asphalt pavement, but using steel slag also can make the temperature of pavement surface increase. In addition, when the BOF slag was heated to a high temperature, the effect of heat energy absorbed by BOF slag was increased. This result implies that BOF slag is capable of storing heat (Xue et al., 2006).

Based on that background, this study will evaluate the performance and thermal properties of porous asphalt using steel slag as aggregates. The objectives of this study are

- To evaluate the performance of porous asphalt containing steel slag.
- To find which type of slag would be better as coarse aggregate in PAC.
- To find the optimum percentage of steel slag as substitutes for coarse aggregates.
- To evaluate the thermal properties of porous asphalt containing steel slag.

2. MATERIALS AND TESTING PROGRAM

The testing program was designated into three parts: basic properties of materials, performance of the PAC mixture, and thermal characteristic of the PAC mixture. Three different percentage of steel slag, incorporating 0%, 30%, and 50%, were considered. It has to be noted that there were resilient modulus (ASTM D4123), skid resistance (ASTM E303), moisture susceptibility (AASHTO T283) included to evaluate PAC mixtures containing steel slags. The thermal characteristics, naming thermal conductivity and heat capacity were assessed by the ISOMET 2104 (Applied Precision Ltd.).

3.1 Chemical and Physical Properties of Steel Slags

Two types steel slag will be used as substitute of coarse aggregate, one Basic Oxygen Furnace (BOF) and other is Baoshan Slag Short Flow (BSSF). The slag was obtain from Dragon Steel Company in Taiwan. The remaining aggregates was using the sources of river gravel. The aggregates shall consist of clean, tough, durable fragments of crushed stone, or crushed gravel of uniform quality throughout. The asphalt binder used in this study was a customized high-viscous polymer modified binder. The asphalt binder was sampled and picked from the Chien-Chung Construction Corporation.

Steel slags primarily consist of SiO2, Al2O3, CaO, and MgO. Also, the high CaO content with a greater affinity for oil than for water can improve the adhesion between the particle and asphalt binder. The contents of f-CaO and MgO are significant factors for the volume expansion potential of slags in asphalt mixtures, but the content of these oxides are depended on raw material resources of the steelmaking procedure.

Data in Table 1 show the chemical composition in comparison with BOF and BSSF slags, this data was obtained from Dragon Steel Corporation. The result show that the content of f-CaO in BSSF slag is significantly lower the content of f-CaO in BSSF slag is significantly lower. This is because in BSSF process, the hot molten slag is quenched by cold water so the f-CaO undergo with the hydrolysis process. The results also show that Ca/Si ratio of BOF slag is higher than BSSF slag, the Ca/Si ratio of BOF slag is range between 4.5 – 5 and the Ca/Si ratio of BSSF slag is range between 4.2 – 4.8. Ca/Si (CaO/SiO2) ratio is used to indicate the content of calcium silicate hydrate (C-S-H), this component has a strong influence on the physical and mechanical properties of materials. In addition, BOF and BSSF slags exhibit strong durability that LA abrasion loss were 20.9 and 21.0, while absorption test results were 2.33% and 1.75%, respectively.

Table 1. Chemical composition of BOF slag and BSSF Slag
2.2 Physical Properties of Asphalt Binder

Polymer Modified Binders (PMB) have widely used in the production of PAC mixtures in Taiwan for improving mixture durability and resisting raveling. The properties of SBS polymer modified asphalt binder were tested at the asphalt laboratory at National Central University and test results are shown in Table 2. To establish mixing and compaction temperatures it is necessary to develop a temperature viscosity chart. From Figure 1, we can get the recommendation temperature for mixing is from 184°C to 194°C and recommendation temperature for compaction is from 172°C to 178°C.

<table>
<thead>
<tr>
<th>Components</th>
<th>CHEMICAL COMPOSITION (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>BSSF</td>
</tr>
<tr>
<td>CaO</td>
<td>33-45</td>
</tr>
<tr>
<td>SiO₂</td>
<td>8-11</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>25-30</td>
</tr>
<tr>
<td>MnO</td>
<td>2.5-3</td>
</tr>
<tr>
<td>MgO</td>
<td>5.5-6</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>1.5-2</td>
</tr>
<tr>
<td>P₂O₅</td>
<td>2-3</td>
</tr>
<tr>
<td>F-Ca</td>
<td>2-6</td>
</tr>
</tbody>
</table>

Table 2. The Properties of polymer modified asphalt binder

<table>
<thead>
<tr>
<th>PROPERTIES</th>
<th>Test results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity (g/cm³)</td>
<td>1.033</td>
</tr>
<tr>
<td>Penetration (25°C, 1/10 mm)</td>
<td>42</td>
</tr>
<tr>
<td>Viscosity (60°C, poise)</td>
<td>147000</td>
</tr>
<tr>
<td>Viscosity (135°C, cSt)</td>
<td>26.4</td>
</tr>
</tbody>
</table>

3. MIX DESIGN

3.1 Aggregate sieve analysis

The sieve analysis of the steel slag samples revealed that the material was a combination of both fine aggregates and coarse aggregates. Figure 2 shows the results of sieve analysis of BOF and BSSF slag when compared with crushed stone. The sieve analysis results show that BSSF slag has finer gradation compared with BOF slag and natural aggregate’s gradation.

3.2 Mix design of PAC with steel slags

Figure 2 Sieve analysis of aggregates used
The porous asphalt mix design was performed based on the local design requirements. This method uses the data from sieve analysis of aggregate to design of the job mix formula. The designed gradation was determined by evaluating three trial-blended gradations. The gradations were classified as upper, middle, and lower. It has to be noted that the designated porosity for PAC was 15% to 25%. The asphalt draindown test (ASTM D6930) was also needed to determine the optimum asphalt content of PAC mixtures.

Table 3 PAC mixtures designed with steel slags

<table>
<thead>
<tr>
<th>Sieve</th>
<th>JOB MIX FORMULA DESIGNED</th>
<th>Specification (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>inch</td>
<td>mm</td>
<td>0% slags</td>
</tr>
<tr>
<td>1&quot;</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>97</td>
<td>97</td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>77</td>
<td>77</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>54</td>
<td>54</td>
</tr>
<tr>
<td>No.4</td>
<td>19</td>
<td>19</td>
</tr>
<tr>
<td>No.8</td>
<td>12</td>
<td>12</td>
</tr>
<tr>
<td>No.16</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>No.30</td>
<td>7</td>
<td>7</td>
</tr>
<tr>
<td>No.50</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>No.100</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>No.200</td>
<td>3.6</td>
<td>3.6</td>
</tr>
</tbody>
</table>

Three JMFs of PAC containing slags were more and less identical with that of PAC without containing slag. It was to achieve same structural characteristic that enables to evaluate the effects of adding slags in PAC. Table 3 shows the blended gradation for each PAC mixtures designed. It has to be noted that it was also to design the PAC mixture containing 50% BSSF. However, the trial batches in the laboratory unveiled that it was not able to achieve the same JMF and structural characteristic as designed. Hence, the further testing program was not including the PAC mixture containing 50% BSSF slags.

4. TEST RESULTS AND ANALYSIS
4.1 Resilient Modulus

In the indirect repeated load testing, the resilient modulus is determined using the recoverable horizontal and vertical deformations that occur during the unloading portion of the load-unload cycle. The test is normally performed over a range of temperatures and stresses to simulate moving vehicles over a pavement structure (e.g., surface, subbase, and subgrade) during the service life of the asphalt pavement. Resilient modulus testing was carried out in accordance with ASTM D4123. Three variations of temperature were carried out, naming 10°C, 25°C and 40°C. The resilient modulus test results for each PAC mixtures are shown in Figure 3.
The effect of steel slag as aggregates substitution on resilient modulus of PAC was investigated in this study. The results of resilient modulus at 25°C were not investigated because the results did not give a significant difference. The results show that increasing the percentage of steel slag increase the resilient modulus of PAC, however as the temperature increases, the resilient modulus values significantly decrease regardless of the steel slag contents. In addition, it also can be found that resilient modulus of PAC containing BOF at 10°C are significantly higher than that of PAC containing BSSF.

4.2. Skid Resistance

Skid resistance is the force developed when a tire that is prevented from rotating slides along the pavement surface. In this study, British Pendulum Tester was used to measure skid resistance of a PAC surface. The British Pendulum Number (BPN) is the mean of five readings or the constant of three readings. As the stiffness of the rubber slider will vary with temperature a correction has to be made if the temperature is not 20°C. Values of BPN for different surfaces of porous asphalt pavement with 30% and 50% of steel slags are shown in Table 4.

<table>
<thead>
<tr>
<th>PAC mixtures</th>
<th>Surface Temperature, °C</th>
<th>BRITISH PRNDULUM NUMBER</th>
<th>Average</th>
<th>BPN (C20)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0%</td>
<td>24.8 54 54 55 56 55</td>
<td>54.8</td>
<td>55.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>25.4 53 56 60 54 55</td>
<td>55.6</td>
<td>58.9</td>
<td></td>
</tr>
<tr>
<td></td>
<td>25.2 55 58 56 57 56</td>
<td>56.4</td>
<td>59.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>25.4 56 57 60 55 58</td>
<td>57.2</td>
<td>60.5</td>
<td></td>
</tr>
<tr>
<td>BOF 30%</td>
<td>27.2 55 54 55 55 53</td>
<td>54.4</td>
<td>58.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>27.2 56 54 55 54 55</td>
<td>54.8</td>
<td>59.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>27.0 59 60 56 57 56</td>
<td>57.6</td>
<td>61.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>27.4 55 56 56 57 56</td>
<td>56.0</td>
<td>60.4</td>
<td></td>
</tr>
<tr>
<td>BOF 50%</td>
<td>26.8 54 50 55 54 55</td>
<td>53.6</td>
<td>57.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>26.6 52 52 51 52 53</td>
<td>52.0</td>
<td>55.9</td>
<td></td>
</tr>
<tr>
<td></td>
<td>26.8 55 54 55 55 55</td>
<td>54.8</td>
<td>58.8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>26.6 52 53 53 53 54</td>
<td>53.0</td>
<td>56.9</td>
<td></td>
</tr>
<tr>
<td>BSSF 30%</td>
<td>28.0 55 53 54 53 50</td>
<td>53.0</td>
<td>57.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>27.6 50 50 52 53 52</td>
<td>51.4</td>
<td>55.9</td>
<td></td>
</tr>
<tr>
<td></td>
<td>28.1 52 51 50 50 51</td>
<td>50.8</td>
<td>55.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>27.9 50 52 49 51 51</td>
<td>50.6</td>
<td>55.2</td>
<td></td>
</tr>
</tbody>
</table>

In this study, the effect of steel slag as aggregates substitution on skid resistance of PAC was investigated using British Pendulum Test. It is observed that using steel slag as aggregate substitution may increase the skid resistance of PAC mixtures, however the skid resistance will decrease at certain percentage of aggregate substitution. From this research, we can conclude that PAC containing 30% BOF slag substitution has the best skid resistance than others. Further research
should be done to find the optimum percentage of steel slag on skid resistance of PAC. The enhancement in skid resistance is attributed to angularity and rough surface texture of steel slag. Steel slag also has higher absorption than natural aggregates. An aggregate with a higher percent absorption may have a higher porosity, which may provide an irregular surface. As the aggregate surface is worn down, the irregular surface is renewed providing greater frictional resistance. It has to be noted that the frictional resistance of PAC containing BOF slag was slightly better than that of PAC containing BSSF.

4.3 Moisture susceptibility

Moisture produces the loss of adhesion between asphalt binder and aggregate surface, and accelerates the development of distresses such as potholes, cracking and raveling. Moisture susceptibility tests were carried out in accordance with AASHTO T283 procedures. The moisture susceptibility is quantified by comparing the average indirect tensile strength of conditioned subset with that of the average of control subset. For the TSR calculation results of PAC shown in Table 5.

Table 5. TSR of PAC mixtures

<table>
<thead>
<tr>
<th>PAC mixtures</th>
<th>Control</th>
<th>After conditioning</th>
<th>TSR (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0% slag</td>
<td>753</td>
<td>676</td>
<td>89.9</td>
</tr>
<tr>
<td>30% BOF slag</td>
<td>814</td>
<td>761</td>
<td>93.5</td>
</tr>
<tr>
<td>50% BOF slag</td>
<td>834</td>
<td>716</td>
<td>85.9</td>
</tr>
<tr>
<td>30% BSSF slag</td>
<td>862</td>
<td>764</td>
<td>88.7</td>
</tr>
</tbody>
</table>

Moisture susceptibility of pavement is one of the concern for the application of steel slag as aggregate substitution. This is because the content of free CaO and free MgO in steel slag. If the free oxides of calcium (CaO) and or magnesium (MgO) come in contact with water, their hydrated products form tufa. Tufa is a spongy, porous mineral precipitate which tends to be fragile and crumble easily. However the results show that TSR for all of the mixtures exceeded the minimum values of 80% requirement which indicates every mixtures may have sufficient resistance against moist-induced damage.

This results prove that using steel slag as aggregate substitution with correct percentage can improve the moisture susceptibility of the PAC. The enhancement in stripping resistance may be caused by its rough surface pores allows penetration of the asphalt binder to form a strong bond characteristic to improve the adhesion performance between asphalt binder and aggregate surface. However, the application of high percentage of steel slag may decrease the moisture susceptibility of PAC because the content of free CaO in steel slag. The hydrated products of free CaO form tufa, and this mineral will decrease the moisture susceptibility of PAC. From this results, we can conclude that to increase moisture susceptibility of PAC, low percentage of steel slag as aggregates substitution should be done.

4.4 Thermal Properties

The thermal conductivity, heat capacity and thermal diffusivity measured for each type of aggregate substitution in this study. The results show that thermal conductivity of PAC without slag is varied from 0.73 to 1.24 W/m.K; thermal conductivity of PAC containing 30% of BOF slag is varied from 0.78 to 1.60 W/m.K; thermal conductivity of PAC containing 50% of BOF slag is varied from 0.6 to 1.05 W/m.K; and thermal conductivity of PAC containing 30% of BSSF slag is varied from 0.47 to 1.08 W/m.K.

Volume heat capacity measured from each treatment show insignificant difference. The heat capacity of PAC without slag is varied from $0.32 \times 10^5$ to $0.68 \times 10^5$ J/m$^3$K; the heat capacity of PAC containing 30% of BOF slag is varied from $0.26 \times 10^5$ to $0.66 \times 10^5$ J/m$^3$K; the heat capacity of PAC containing 50% of BOF slag is varied from $0.31 \times 10^5$ to $0.79 \times 10^5$ J/m$^3$K, and the heat capacity of PAC containing 30% of BSSF slag is varied from $0.18 \times 10^5$ to $0.56 \times 10^5$ J/m$^3$K.

Thermal diffusivity was calculated by divided the value of thermal conductivity with heat capacity. The results show that thermal diffusivity of PAC without slag is varied from $1.55 \times 10^{-6}$ to $2.42 \times 10^{-6}$ m$^2$/s; thermal diffusivity of containing 30% of BOF slag is varied from $1.84 \times 10^{-6}$ m$^2$/s to $3.19 \times 10^{-6}$ m$^2$/s, thermal diffusivity of containing 50% of BOF slag is varied from $1.30 \times 10^{-6}$ to $2.37 \times 10^{-6}$ m$^2$/s.
10^{-6} \text{ m}^2/\text{s}, and thermal diffusivity of containing 30\% of BSSF slag is varied from 1.59 \times 10^{-6} to 2.78 \times 10^{-6} \text{ m}^2/\text{s}.

Statistical analysis prove that using steel slag as aggregate substitution give significant effect on thermal conductivity and thermal diffusivity of the PAC. Thermal conductivity is the ability or power of materials to conduct or transmit heat. It determines how fast and easily the heat would be conduct from high temperature object/part to low temperature object/part. Pavement with low thermal conductivity may heat up at the surface but will not transfer that heat throughout the other pavement layers as quickly as pavement with higher conductivity. Porous pavement concrete emerging as a potential cool pavement was proven in this study. The results show that PAC without slag has lower thermal conductivity than DGAC. The conductivity of of DGAC range between 1.0 to 1.5 W/mK (Hsu et al, 2012). This mean that dense-compacted materials, such as DGAC, is capable of conducting more heats than that in porous mixtures. However the highest thermal conductivity of PAC containing 30\% of BOF slag is 1.6 W/mK, this value is higher than thermal conductivity of ordinary DGAC.

Figure 4 presents the relationship of the percentage of steel slag substitution with thermal conductivity of PAC. The measured thermal conductivity showed that using steel slag as aggregate substitution may increase thermal conductivity of PAC, however thermal conductivity decrease with the increasing of percentage of steel slag. This is because aggregates is one of the main factor that affect thermal conductivity of the pavement. As we know that steel slag has a higher thermal conductivity compared with the natural aggregate. The highest thermal conductivity value obtained from this study is thermal conductivity of the PAC containing 30\% of BOF slag mixtures.

This results also indicates that there is relation between the pavement surface with thermal conductivity of the pavement. PAC containing 30\% of BOF slag has the highest BPN value compared with others. In Figure 5, it shows the relationship between thermal conductivity and pavement surfaces. We can conclude that the smooth surface showed a higher thermal conductivity compared to that of the rough surface. This is because the pavement with smooth surface will have better surface contact for the probe. Also, the value of thermal conductivity is low due to the presence of aggregates in the rough surface and it takes a longer time for thorough heat transfer.

In summary, PAC containing 30\% of BOF slag has the highest thermal conductivity and thermal diffusivity. This mean that PAC containing 30\% of BOF slag will conduct heat and achieve thermal equilibrium faster than other pavements. This is because BOF slag has higher thermal conductivity than natural aggregates and PAC containing 30\% of BOF slag has rougher pavement surface than others.

![Graph](image-url)

Figure 4. The relation of thermal conductivity with the percentage of BOF substitution
5. CONCLUSIONS

Based on the statistical and other analysis of results in this study, the following conclusions can be made:

- BOF slag and BSSF slag can be used as coarse aggregates substitution in PAC. The performance results show that using BOF and BSSF slag as aggregate substitution may improve the performance of PAC, such as improve resilient modulus, skid resistance, and moisture susceptibility. However using high percentage of steel slag as aggregate substitution may decrease skid resistance, rutting potential, and moisture susceptibility of PAC.
- BOF slag is better than BSSF slag as aggregate substitution. With the same percentage, the PAC containing BOF slag has better resilient modulus at 10 °C, skid resistance, rutting potential, and moisture susceptibility than PAC containing BSSF slag.
- Statistical analysis prove that the optimum percentage of steel slag substitution in PAC is 30% of BOF slag. PAC containing 30% BOF slag has the best performance compared others.
- Using BOF and BSSF slag as aggregate substitution may increase thermal conductivity and diffusivity of PAC. The results show that PAC containing 30% of BOF slag has the highest thermal conductivity and thermal diffusivity compared with others mixtures, the maximum thermal conductivity obtained was higher than thermal conductivity of ordinary DGAC.
- The results from this study indicates that increasing the percentage of aggregate substitution may increase the performance of PAC, however the performance will decrease at the certain percentage. Study with the smaller range of aggregate substitution should be conducted to find the optimum percentage of steel slag as aggregates substitution on each performance test.

6. ACKNOWLEDGEMENT

The authors like to express their gratitude to colleagues at the Department of Civil Engineering of National Central University at Taiwan, acknowledge the Double Degree Program of University of Brawijaya at Indonesia, and “Beasiswa Unggulan DIKNAS” to provide the scholarship for this study.

7. REFERENCES

The 1st IRF Regional Congress of the International Road Federation

Pilot surveys of measuring road roughness using a smartphone in Papua New Guinea

Authors:
Mr. Dumitru Ceban, Finnish Overseas Consultants (FinnOC) Ltd., Highway Engineer
ceb.dumitru@gmail.com
Mr. Steven Sapalo, Acting Project Director - World Bank (DoW)
ssapalo@works.gov.pg
Mr. Bien Kaul, Acting Manager (Maintenance & Coordination), Asset Management (DoW)
bkaul@works.gov.pg
Mr. Nickson Laime, Nikki and Associates Ltd. Managing Director
nmlaime@yahoo.co.uk
Mr. Gregory Ume, Finnish Overseas Consultants (FinnOC) Ltd., Liaison Officer
gregory.ume@hotmail.com

INTRODUCTION

The International Roughness Index (IRI) is a roughness index commonly obtained from measured longitudinal road profiles. Since its introduction in 1986, IRI has become commonly used worldwide for evaluating and managing road systems. Vibrations have been used since early 1900 for expressing road condition and ride quality [1]

The traditional techniques for measuring roughness may be categorized as special built trucks or wagons with laser scanners, bump-wagons, and manually operated rolling straight edges. Special built equipment is expensive, due to heavy and complex hardware, low volume of production and need of sophisticated systems and accessories. Data gathering and analysis are often time consuming. Data collection is typically done during the summer then analyzed and delivered to the maintenance management systems in late autumn [2].

As described in Sawyers et al. 1986a, it is necessary to understand the difference between four generic classes of road roughness measuring methods in use [2]:

- Class 1 - Precision profiles
- Class 2 - Other profilometric methods
- Class 3 - IRI by correlation
- Class 4 - Subjective ratings

A smartphone-based system is also an alternative to Class 4 – Subjective rating [3], on roads where heavy, complex and expensive equipment is impossible to use, and for bicycle roads. The technology is objective, highly portable and simple to use. This gives a powerful support to road inventories, inception reports, tactical planning and program analysis, and support maintenance project evaluation.

It should be mentioned that smartphone based systems as Roadroid might challenge old knowledge, standards, procedures and existing ways to procure:
- Pavement planners and road engineers know existing inputs;
- Research organizations, suppliers and buyers have existing ways to work;
- Organizations have invested time, prestige and huge amounts of money to develop more and more exact and complex data collection and management systems.

This paper aims to provide more insight to extensive application of a smartphone-based system for measuring roughness in a number of projects carried out by FinnOC ltd in Papua New Guinea during period of January – July 2014 and was seen as a potential:
- Tool for road roughness measurements and obtaining current condition data of individual road sections, including those in remote areas.
- Tool for quick build-up and update of PNG national road network database.

The scope of the study was identification of cIRI correlation factors for several vehicle models used in surveys, especially Toyota Land Cruiser and Toyota Hilux that are the major vehicle fleet of PNG regional road authorities. The applicability as a universal roughness measurement tool was also investigated. The need of performing the experiment appeared after consulting relevant literature. The Roadroid equipment is an innovative approach in measuring IRI, developed to its current shape in 2013 and is available to wide public since late 2013.

After consulting the technical data provided by the manufacturer it became obvious that the technology is in process of constant development and more detailed data might be available at a later stage. However, most of studies have been carried out in Sweden. Limited studies that have been carried out in developing countries do not correlate the measured data with a class I reference profile/profilometer.

Roadroid is a modern system which allows monitoring the road conditions with a smartphone. It has a good accuracy and can easily be mounted on several different types of vehicles. The system is user friendly and durable. The concept is continuously being developed hand in hand with the evolving mobile technology. The decision of using this equipment was influenced by its competitive cost, portability and ease of operation. The manufacturer’s specification sheets specify that Roadroid is a measurement system that falls in the class III of precision. The stated accuracy is about 80%.

With the following characteristics, the system appeared very attractive from operational point of view in conditions of Papua New Guinea, where the road network has a number of links in very poor condition where a robust and practical measurement device is required and what is important can be mounted easily on most of cars and jeeps. The outputs are generated easily within 24 hours without much processing.

**SETUP**

The same Roadroid device has been used throughout all study. The set comprised a Smartphone (in this case it was a Samsung i9105) with Roadroid software installed, a phone rack for mounting on windshield, and a charging cable.

To be noted that time required for preparing the Roadroid device for surveys was limited to 5-7 minutes. Comparing Roadroid with Roughometer III and ROMDAS, the Roadroid shows a clear advantage.

The Roadroid device was used in surveys on mainly two types of vehicles: Toyota Land Cruiser and Toyota Hilux and always installed in the middle of the windshield (see Figure 1). Both vehicle types are widely used by PNG Road Authorities, including but not limited to road surveys and road inspections. Although in similar in vehicle class, they vary significantly in physical characteristics. Based on discussion with local road authorities it was assumed that Toyota Land Cruiser’s suspension is softer than Toyota Hilux’s one. The latter was observed to be generally stiffer (regardless of vehicle year of production and type of body) in all surveys carried out by FinnOC ltd.

Figure 1. Example of mounting the Roadroid device in the vehicle.
The manufacturer recommends collecting cIRI values. cIRI values express the road roughness based on quarter car suspension model.

In order to produce more accurate results, the Roadroid must be “calibrated” by choosing one of the following settings:

“Vehicle type is mentioned, and lets you set a) Small car/business van as Renault Kangoo or b) a Medium/big sedan/station wagon or estate as = Volvo v70 or c) a 4WD jeep type = as Toyota Hilux or Nissan King Cab.” [4].

Another setting that influences survey results based on the type of specific vehicle and/or vehicle class is the cIRI adjustment constant.

“The cIRI Adjustment constant is used to adapt the sensitivity of the cIRI algorithm. cIRI actually calculates the IRI with the original quarter car formula (while cIRI is estimating it from the vibrations). The adjustment option will enable to do data sampling on a road where you have a known reference (maybe a test or calibrations section you use). The constant goes between 0.5-4 and the default is 1.5. Currently you need to upload your data, get the raw or 100 meter sectioned text file and import to excel and compare with your reference to adjust your setting.” [4]

For both vehicles the vehicle type, the setting was set to a 4WD jeep type. The cIRI adjustment constant was set to 1.5 and 1.0 for Toyota Hilux subsequently. The cIRI adjustment constant 1.0 for Toyota Hilux is based on the field observations. To be noted that cIRI adjustment constant of 1.0 was the smallest value in the latest software version of Roadroid available at the time of surveys. It was observed that on roads in worse condition the software will consistently reset when hitting stronger bumps. The device would stop recording and would ask for recalibration. Also the maximum travel speed was reduced to 40 km/h where possible in order to avoid exaggerated response from the vehicle suspension.

SURVEYS

During period of January to July 2014 about 1,160 lane-km were surveyed by FinnOC ltd engineers. The road sections of PNG road network were surveyed using Roadroid equipment are presented in Table 1. The numbers in brackets represent the areas on the map (see Figure 2).

The studies carried out by FinnOC and associates comprised a number of roads with representative surfaces, geometric layout and as well as different road classes.

The inspected roads vary in surface type from earth to asphalt. Geometric alignments vary from straight and level coastal plains to mountainous roads with numerous serpentines. The studied road sections were part of different road classes from National Highways to District Roads.

Table 1. Road links surveyed in PNG during January-July 2014

<table>
<thead>
<tr>
<th>Link name</th>
<th>Province</th>
<th>Length, km</th>
<th>Road type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Laloki Bridge – Port Moresby</td>
<td>National Capital District (1)</td>
<td>7.5</td>
<td>Paved</td>
</tr>
<tr>
<td>Togoba jct – Mt. Hagen</td>
<td>Western Highlands (2)</td>
<td>24*</td>
<td>Paved</td>
</tr>
<tr>
<td>Alotau – East Cape</td>
<td>Milne Bay (3)</td>
<td>56</td>
<td>Unpaved</td>
</tr>
<tr>
<td>Alotau – Gadaïs village</td>
<td></td>
<td>121*</td>
<td>Mixed</td>
</tr>
<tr>
<td>Port Moresby – Kerema</td>
<td>Central, Gulf (4)</td>
<td>297*</td>
<td>200 km Paved</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>100 km Unpaved</td>
</tr>
<tr>
<td>Madang- Awar Bridge</td>
<td>Madang (5)</td>
<td>230</td>
<td>Unpaved</td>
</tr>
<tr>
<td>Beon – CIS road</td>
<td></td>
<td>6</td>
<td>Mixed</td>
</tr>
<tr>
<td>Nubia – Base Camp</td>
<td></td>
<td>20**</td>
<td>Earth road</td>
</tr>
<tr>
<td>Highland Highway – Boana Station</td>
<td>Morobe</td>
<td>32</td>
<td>Unpaved</td>
</tr>
</tbody>
</table>

Note: * recorded both ways
** recorded partially due to very poor condition

The following figure indicates the areas where the surveys were carried out.

![Map of Papua New Guinea showing survey locations](image)

**Figure 2. Project location map of Papua New Guinea indicating location of surveys.**

**DATA VALIDATION**

In order to verify the consistency and accuracy of collected data and to find the most appropriate correlation factor (cIRI adjustment constant), the FinnOC Ltd and associates, in cooperation with Head Office of Department of Works of PNG (DOW) have performed a series of test surveys on a conventional test site. DOW identified the reference road segment. The reference test site represented a stretch of road in good condition with a cape seal surface (see Figure 2). The geometry is straight with a very slight longitudinal grade and 300 m long. The “real” roughness of the reference portion of the road was measured in advance with a Class 1 profilometer – ROMDAS Z250 dipstick. The resulted average IRI index measured by ROMDAS Z250 was equal to 2.5 IRI.

The validation process was planned to carry out a set of surveys to track the repeatability as well as the correlation to the real average IRI of the test site. Therefore:

- 4 runs were performed with the same cIRI adjustment constant chosen arbitrarily;
- 6 runs were performed with Toyota Hilux and cIRI adjustment ranging between 0.8 – 1.5;
- 7 runs were performed with Toyota Land Cruiser and cIRI adjustment ranging between 2.0 – 3.75.

![Reference site](image)

**Figure 3. General view of the reference site.**
Tables 2 and 3 summarize the runs, the estimated and aggregated eIRI values generated by Roadroid, cIRI aggregated values calculated by Roadroid as well as cIRI adjustment constant and the correlation factors to bring the values to IRI measured by Class 1 profilometer. Table 2 contains values measured by device installed on Land Cruiser and Table 3 shows values for Toyota Hilux. Table 4 summarizes the four runs with Toyota Hilux.

Table 2. The measured and computed values using Land Cruiser as a survey vehicle.

<table>
<thead>
<tr>
<th>Run</th>
<th>cIRI adj. constant</th>
<th>eIRI</th>
<th>cIRI</th>
<th>IRI factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.0</td>
<td>2.05</td>
<td>1.82</td>
<td>1.37</td>
</tr>
<tr>
<td>3</td>
<td>2.5</td>
<td>1.86</td>
<td>2.79</td>
<td>0.90</td>
</tr>
<tr>
<td>4</td>
<td>2.75</td>
<td>1.83</td>
<td>2.53</td>
<td>0.99</td>
</tr>
<tr>
<td>5</td>
<td>3.0</td>
<td>1.74</td>
<td>3.18</td>
<td>0.79</td>
</tr>
<tr>
<td>6</td>
<td>3.25</td>
<td>2.19</td>
<td>4.34</td>
<td>0.58</td>
</tr>
<tr>
<td>7</td>
<td>3.5</td>
<td>1.99</td>
<td>3.79</td>
<td>0.66</td>
</tr>
</tbody>
</table>

Table 3. The measured and computed values using Hilux as a survey vehicle.

<table>
<thead>
<tr>
<th>Run</th>
<th>cIRI adj. constant</th>
<th>eIRI</th>
<th>cIRI</th>
<th>IRI factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1*</td>
<td>0.8</td>
<td>3.86</td>
<td>1.28</td>
<td>1.95</td>
</tr>
<tr>
<td>2</td>
<td>0.9</td>
<td>4.19</td>
<td>1.32</td>
<td>1.88</td>
</tr>
<tr>
<td>3</td>
<td>1.0</td>
<td>5.02</td>
<td>1.95</td>
<td>1.28</td>
</tr>
<tr>
<td>4</td>
<td>1.1</td>
<td>4.11</td>
<td>1.84</td>
<td>1.35</td>
</tr>
<tr>
<td>5</td>
<td>1.2</td>
<td>4.84</td>
<td>2.18</td>
<td>1.14</td>
</tr>
<tr>
<td>6</td>
<td>1.3</td>
<td>5.18</td>
<td>2.51</td>
<td>0.99</td>
</tr>
<tr>
<td>7</td>
<td>1.5</td>
<td>4.44</td>
<td>2.84</td>
<td>0.88</td>
</tr>
</tbody>
</table>

Note: * Average of four runs with 0.8 cIRI adjustment constant

Table 4. The measured and computed values using Hilux as a survey vehicle.

<table>
<thead>
<tr>
<th>Run</th>
<th>cIRI adj. constant</th>
<th>eIRI</th>
<th>cIRI</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.8</td>
<td>4.03</td>
<td>1.59</td>
</tr>
<tr>
<td>2</td>
<td>0.8</td>
<td>2.08</td>
<td>0.80</td>
</tr>
<tr>
<td>3</td>
<td>0.8</td>
<td>4.93</td>
<td>1.40</td>
</tr>
<tr>
<td>4</td>
<td>0.8</td>
<td>4.39</td>
<td>1.34</td>
</tr>
</tbody>
</table>

Figure 4. The roughness profile of four runs, aggregated to 25m.
From the results can be observed that from four repeatability runs on Toyota Hilux, based on aggregation interval of 25m, the average value is 3.86 eIRI. The standard deviation of eIRI based on average of four runs is 1.24. The computed IRI (cIRI) average is 1.28 and standard deviation of 0.33.

The standard deviation of cIRI 1.28 represents 26% of the average cIRI based on four runs and aggregated to 25m interval.

Considering eIRI based on 10 runs (the value related to vibrations that cannot be influenced by user) the resulting average value is 4.32. The standard deviation of eIRI based on total number of runs, aggregated to 25m interval, equals to 2.04. The minimum value of all readings is 1.62 and the maximum is 11.14. The figure below shows distribution of aggregated 25m readings based on ten runs. From figure could be observed that 93 samples out of 118 are situated within ±1 Standard Deviation (sdev) from median value.

![Distribution of samples](image)

**Figure 5. Distribution of readings.**

From the results can be observed that from 7 runs with Toyota Land Cruiser, the average eIRI (the value related to vibrations that cannot be influenced by user) of seven runs equals to 1.94 eIRI. The standard deviation of eIRI based on 7 averaged runs equals to 0.15, which represents 7% of average.

The cIRI adjustment constant of 2.75 and 3.0 provide an IRI value of 2.79 and 2.53 measured with Land Cruiser. Closest to the reference IRI is 3.0, whereas the cIRI adjustment value of 1.3 measured with Hilux produces an IRI of 2.52 which is closest to the reference roughness measured by class I profilometer.

CONCLUSIONS

The results show a strong repeatability of IRI. After conducting 10 repetitions, it is visible that every time Roadroid generate similar data within a range of +/- 0.33 units.

The survey proved a significant difference in readings when comparing results from both vehicles. Hence, the “vehicle type” option was the same for all surveys (Hilux/ KING Cab). The vehicle suspension stiffness has strong influence on the IRI readings. The difference is easily observed when comparing two datasets. With different cIRI adjustment constant (i.e. different sensitivity values for different vehicles can produce the same output value.

<table>
<thead>
<tr>
<th>Toyota Hilux</th>
<th>Toyota Land Cuiser</th>
<th>Reference IRI</th>
</tr>
</thead>
<tbody>
<tr>
<td>eIRI</td>
<td>cIRI</td>
<td>IRI</td>
</tr>
<tr>
<td>5.18</td>
<td>2.51</td>
<td>1.83</td>
</tr>
<tr>
<td></td>
<td>2.53</td>
<td>2.5</td>
</tr>
</tbody>
</table>

The table above shows that the final outputs may vary considerably with specific vehicle model if the same cIRI adjustment is used.

The experiment showed that the recommended cIRI adjustment constant value of 1.5 is not always appropriate on visually rough roads. As a guide the road roughness can be assessed according to [5]. Field surveys showed that on rough roads 0.8-1.0 cIRI adjustment values are appropriate for Toyota Hilux.
Considering section length the presented results contain systematic errors [6]. These errors can be eliminated if a longer test section is used. At time of carrying out the experiment only a section of 300m with true measured IRI was available.

Roadroid can be used within a broad range of speeds from 20km/h to 100km/h depending on the road surface. Better the road is, the higher speed can be used in the survey. FinnOC ltd. engineers observed that speeds higher than 70km/h provide inaccurate results on unpaved roads. On roads that are generally in poor condition the speed should be decreased to 40km/h as unexpected software drop-outs can occur due to high shock (bump).

Roadroid data can be compared and based on specific type of vehicle adjusted to reference data based on calibration test site data. Generally a few runs would be enough to validate the data and choose the appropriate cIRI adjustment constant.

- Roadroid as a tool for decision making process:

The Roadroid equipment applicability was roughly compared against Australian developed device Roughometer III and ROMDAS New Zealand developed device both designed for road roughness measurements.

If to compare Roadroid to ROMDAS and Roughometer III the latter require a dedicated vehicle and many more calibration coefficients. Roadroid doesn’t require a dedicated vehicle and can be mounted on virtually any light vehicle. It doesn’t require any special tools for installing the equipment. All these are reflected in shipment, installation and transportation times. Roadroid system is very easy to carry setup and operate due to its extreme compactness. It is physically compact and can fit in a regular backpack.

Conducting repeating surveys of the same site, was observed that Roadroid would produce cIRI within 20-25% of average, which means that system has a good repeatability and therefore will produce reliable results.

It is important to stress that smaller aggregation intervals (usually less than 100m) will produce a bigger variance of values. In practice, data would be aggregated to 100-500m. In this case, the device produces outputs that are more accurate.

The data validation was performed on the specific test site in order to check the Roadroid generated outputs against a class I profilometer. In practice, various road segments have different roughness and a range of wavelengths. The important findings from data validation test are that Roadroid is capable to record reliable data that is very important if used as an objective condition assessment tool; it is desirable to verify the cIRI adjustment constant for particular vehicles if the vehicle is to be used extensively throughout surveys.

The data validation test showed that the appropriate cIRI adjustment constant for Toyota Hilux is 1.3. During surveys though, it was observed that on rough roads this constant is too high and should be lowered at least to 1.0. The speed also has to be reduced to the range of 20-40 km/h in order to record the road surface condition without unexpected interruptions due to severe bump and reset of Roadroid.

No unexpected reset issues were observed when Toyota Land Cruiser was used in surveys and driven with sensible speed according to surface condition (i.e. normally up to 60km/h on unsealed roads and up to 80km/h on paved roads)

Furthermore, the results of data validation test showed that the cIRI adjustment constant is much higher for Land Cruiser than for Hilux which indicates that different suspension stiffness plays a moderate role even within same class of vehicles.

Initially, in earlier surveys using Land Cruiser the recommended default cIRI adjustment constant of 1.5 was used. When results from reference test site measured by class I profilometer was made available, few test runs with different cIRI adj. constant were done with Land Cruiser to find out the difference in readings.
The survey results obtained from Roadroid were further used in order to find the IRI correlation factor. A few subsequent project surveys were then factored using the correlation factors to obtain the IRI.

The raw data is easily uploaded to cloud server. After uploading on server, the user through a login and password has direct access to view the survey on the map as well as direct access to the aggregated data. The aggregated data is downloaded in notepad format which could be accessed on virtually any computer. The data can be directly transferred to MS Excel and processed according to needs.

An important fact is that the system and the cloud server are under constant improvement and new features will be implemented.

Finally could be said that Roadroid meets the requirements for building a comprehensive road management database for future development of PNG road network.

REFERENCES

Evaluation of Moisture Susceptibility of Asphalt Mixture Using Image Analysis and Performance tests

Dae-Wook Park¹, Jun Kim², Vo Viet Hai³, Hyeok-Jung Kim⁴, Jun-Sang Park⁵

Abstract: In this study, the moisture susceptibility of asphalt mixture was evaluated for three different asphalt mixtures which are contained hydrated lime and two anti-stripping agents. For comparison purpose, asphalt mixture without any additives was used to compare the resistance of moisture susceptibility of asphalt mixture contained additives. The indirect tensile strength ratio (TSR) was used to evaluate the moisture susceptibility of asphalt mixture and test specimens were conditioned according to Korea Standard (KS) F 2398 which is same with AASHTO T 283. And the amount of stripping was measured using a digital camera which has 8.1 mega pixels and analyzed using the developed program in this study. The results showed that asphalt mixtures contained anti-stripping agent have significantly higher TSR values than asphalt mixture with hydrated lime limestone filler. The asphalt mixture with hydrated lime showed slightly higher TSR value than asphalt mixtures without any additives with anti-stripping agent. The amount of stripping is closely related to indirect strength by both wet test rather than and the TSR values.

1 INTRODUCTION

Stripping is the loss of bond between the asphalt and aggregate in asphalt concrete structure. The common causes in all cases of stripping are the presence of water. The moisture damage causes loss of adhesion, and adversely affect to the strength of the asphalt mixture dramatically. It can also cause the premature pavement failures such as rutting and raveling on the pavement surface (Taylor and Khosla 1983, Solaimanian et al. 1993). Moisture sensitivity testing has been applied by many agencies in the asphalt mixture design stage and the result of this testing can be used to eliminate certain asphalt and aggregate combinations or to investigate the needs of an anti-stripping additive. The potential to incur moisture damage can be controlled or reduced by material selection, mixture designs, increasing a high asphalt film thickness, additives, proper pavement design, compaction, and drainage (Little and Epps 2001). Solaimanian et al. (2003) suggest that the most common technique to mitigate moisture damage is the use of additives or modifiers with the asphalt binder or the aggregate, and AASHTO T-283 is the most widely recognized laboratory test method for the evaluation of moisture susceptibility. Al-Qadi et al. (2014) summarized that developing laboratory moisture damage evaluation tests is challenging and it is hard to simulate field performance because of the high variability of the factors affecting moisture damage and the process of developing new test procedures still continues. Amelian and Abtahi (2014) concluded that the boiling water test was converted from subjective rating to a more objective evaluation by applying image analysis techniques and there was a good relationship with tensile strength ratio (TSR) values. Almost all the liquid anti-stripping additive and the hydrated lime were effective and improve the moisture susceptibility resistance of HMA mixtures, met or exceeded the tensile strength ratio criteria. Rahman (2012) also proved that the application of hydrated lime in asphalt binder is an effective and economical method to evaluate moisture damage as well as rutting.

The purposes of this study are to evaluate the moisture susceptibility of asphalt mixtures with four types of anti-stripping additive based on indirect tensile strength (ITS) test. The work includes conducting both test methods on specimens prepared from different additives. Digital images captured from wet ITS test were analyzed and compare with ITS and TSR obtained from dry and wet conditioned specimens in KS F 2398 test.

---

¹ Corresponding Author, Associate Professor, Dept. of Civil Engineering, Kunsan National University, San 68 Miryong-dong, Kunsan, Chellabuk-do, 573-701, Korea, E-mail: dpark@kunsan.ac.kr
² Graduate Assistant, Dept. of Civil Engineering, Kunsan National University, San 68 Miryong-dong, Kunsan, Chellabuk-do, 573-701, Korea, E-mail: imjun0524@naver.com
³ Ph.D Student, Dept. of Civil Engineering, Kunsan National University, San 68 Miryong-dong, Kunsan, Chellabuk-do, 573-701, Korea, E-mail: haiwo2310@gmail.com
⁴ Associate Researcher, Kumho R&D Center, #1557 Yuseong-Dae Ro, Yuseong-gu, Daejeon, 305-348, Republic of Korea, E-mail: ceesare@kkpc.com
⁵ Chief Researcher, Kumho R&D Center, #1557 Yuseong-Dae Ro, Yuseong-gu, Daejeon, 305-348, Republic of Korea, E-mail: polymer@kkpc.com
2 MATERIALS

Asphalt binder and aggregate are the main components in asphalt mixture. The aggregates represented materials currently in production for hot mix asphalt with an addition of recycle asphalt pavement (RAP) of 30 percent. The combined aggregates and RAP were used to meet the required aggregate distribution. Asphalt binder PG58-22 grade was used as controlled or original binder.

The four additives limestone filler (LF), hydrate lime (HL), two anti-stripping agents; “K” anti-stripping agent made in Kumho Chemcical R&D Center and “A” anti-stripping agent named Well-fix were used. In all cases the mineral additives hydrate lime and limestone filler were dosed at a rate of 2 percent by weight of aggregates. The liquid additives were dosed at rates of 0.5 percent by weight of new asphalt binder.

3 SAMPLE PREPARATION

To prepare samples, aggregate is first placed in the oven for more than 4 hours, and less than 1 hour for asphalt binder. Enough material is mixed to produce six specimens at the 4.1 percent of asphalt binder of each mixture. After mixing, the mixture is placed in the pans and spread; then cooled to room temperature for 2 hours. After that the asphalt mixture is placed in the oven for 24 hours at 60 Celsius degree and it is heated to compaction temperature before 2 hours of compaction. All the asphalt mixture specimens were compacted at air voids of 7 ± 0.5 percent.

4 TESTING

Indirect tensile strength (ITS) test was conducted to investigate the effect of anti-stripping additives on moisture susceptibility characteristic. The damage due to moisture is controlled by the specific limits of the tensile strength ratios (TSR). ITS test was performed according to the KS F 2398 by compacted specimens of 100 mm in diameter and 63.5 mm in height at air voids of 7 ± 0.5 percent. The specimens are separated into two subsets, one subset for dry ITS test and the other subset for wet ITS test. The test procedure is summarized as shown in Figure 1.

During the test, the compressive load was applied through two loading strips. The maximum indirect tensile force was recorded and the corresponding ITS of the asphalt mixture was determined. The tensile strength ratio (TSR), a ratio of the ITS of wet conditioned specimens to the ITS of dry specimens, was calculated and used as a moisture susceptibility index of asphalt mixtures. The ITS is calculated using the following equation:

\[ S_i = \frac{2000P}{\pi Dt} \]  

where:  
\( S_i \) = the indirect tensile strength (kPa)  
\( P \) = the maximum load (N),  
\( t \) = thickness of specimen (mm)  
\( D \) = diameter of specimen (mm).

The tensile strength ratio (TSR) is determine from the dry and wet ITS test results:

\[ TSR = \frac{S_{w}}{S_{d}} \]  

where:  
\( S_{w} \) = average wet ITS (kPa), and  
\( S_{d} \) = average dry ITS (kPa).
Figure 1. Dry and wet ITS test diagram

Two half of the samples after conducting wet ITS test were kept for image stripping analysis. The images were captured to the broken surface by digital camera (SONY DSC-T100 8.1 mega pixels) in an adequate and indirect light condition to prevent creation of light reflection. It is recommended to use a light green color as background during taking picture. With this color, background separation can be done accurately and evenly. Moreover, the angle of images capturing should be perpendicular to the surface of sample. The example of recommended image outline is shown in Figure 2(a).

The image analysis program developed by Matlab (2013) was employed for determining amount of stripping. The images in JPEG format were first imported into the program. After that, the series of analyses were implemented as follow:

- The sample colors were segmented, detected and separated from the background color.
- The color images were converted to binary images using global thresholding.
- The numbers of black and white pixels were counted to determine an amount of stripping.
- The area of sample surface determined, including stripping.
- Stripping percentage was calculated by the ratio of stripping to surface sample area.

According to the observation, any nonblack areas should be considered stripping; therefore, different global threshold levels were applied to images and then the threshold value of 0.165 was chosen to distinguish the white and black pixels, the program identifies the stripping areas as white pixels. This threshold level seems to be a realistic value to recognize the amount of stripping. Applying different threshold levels indicated that the global threshold values between 0.16 and 0.17 did not significantly affect the stripping percentage results; however, for an accurate comparison, it is important to use the same threshold value for all tests. The example image after analyzing is shown in Figure 2.
Figure 2. Example of image analysis; (a) image of specimen, (b) image of sample detection and separation, (c) image of sample area, and (d) image of stripping area

5 ANALYSES

According to Figure 3, the asphalt concrete mixtures containing the additives LF, HL, K, A and controlled asphalt have TSR values of 55.4%, 88.3%, 74.3%, 82%, and 52.4% respectively. Accordingly, there are two mixtures HL and A meet the criteria of a minimum TSR value of 80% (KS 2012), but the TSR values of all asphalt mixtures with additive are higher than that of asphalt mixture with controlled asphalt which is without anti-stripping additives. Basically, the additives have positive effects on performance and moisture susceptibility of asphalt concrete mixture. Krishmankuttyair (2008) found that the values of dry and wet ITS together with TSR values should all be employed to evaluate the effect of water damages on performance of asphalt concrete mixtures. For instance, the asphalt mixture with HL has a low dry and wet ITS values, however, as seen in Figure 3(b), the moisture susceptibility or TSR value has been improved significantly as compared to the others.
Figure 3. (a) Indirect tensile strength (ITS) and (b) Tensile strength ratio (TSR) results

In image analysis test, the lower the amount of stripping, the less the moisture damage occurs. In Figure 4, the results indicate that the amount of stripping is closely related to indirect strength by both wet ITS test and TSR values with the exception of the mixture with HL in ITS test. It can be attributed the image analysis is basically reliable when its results, amount of stripping, have almost the same trend with the wet ITS test and TSR values. Even though the ITS test result of the asphalt mixture with HL is low, but the image analysis indicates a good resistance to stripping (low amount of stripping), match with the high TSR value.
6 CONCLUSIONS

The evaluation of stripping resistance of asphalt mixtures with various additives was conducted based on the TSR value and aggregate stripped surfaces in asphalt samples were quantified using digital image analysis. In this study, an improved stripping resistance was investigated on asphalt mixtures containing additives. In particular, the highest resistance was obtained for asphalt modified with anti-stripping agents; in the case of hydrate lime, the presence of the hydrate lime probably reduced indirect tensile strength but improved the moisture susceptibility. Image analysis indicated a good relationship between its results and indirect tensile test results, ITS and TSR values. Moreover, the result of image analysis can be applied or use in further research for verifying and evaluating stripping in certain cases.

REFERENCES

Rahman, S. (2012). The way to resist moisture damage and rutting in asphalt mixture in Bangladesh by the application of hydrated lime. Journal of Mechanical and Civil Engineering, Vol. 3, 36-40
PAPER TITLE
(90 Characters Max)
WARM MIX ASPHALT FOR HEAVY TRAFFIC IN INDONESIA

TRACK

AUTHOR
(Capitalize Family Name)
Furqon AFFANDI

POSITION
Senior Researcher

ORGANIZATION
Institute of Road Engineering (IRE)

COUNTRY
Indonesia

CO-AUTHOR(S)
(Neny KUSNIANTY)

E-MAIL
(for correspondence)
furqon_affandi@yahoo.com

KEYWORDS:
mixing temperature, Warm Mix Asphalt, additive, emission; rutting resistance.

ABSTRACT:
Indonesia has implemented the specification of hot mix asphalt (HMA) for heavy traffic, associated with increasing traffic load as well as high temperature and pavement in Indonesia, according to its location in equator. HMA for heavy traffic using modified asphalt requires mixing temperatures in AMP around 175-180 °C. This creates difficulties because of mixing temperature is higher than conventional HMA (asphalt pen 60) and will require more fuel and generate higher emissions. Warm Mix Asphalt (WMA) by adding some additive to asphalt 60 pen grade, can reduce mixing and compaction temperatures of about 40 °C, compared to HMA for heavy traffic with quality equivalent to HMA using modified asphalt and meet specified requirements of mixture for heavy traffic. Rutting resistance of WMA is bigger around 1.89 times than HMA, which indicated by dynamic Stability of WMA and HMA is 5200 passing / mm and 2742 passing / mm respectively. Resistance to moisture induce damage of WMA and HMA meet the requirements which greater than 80%. Resistance to material loss tested by Cantabro method (AASHTO D??? xxx) showed good results, where its material loss is only 2% and 3% respectively for WMA and HMA which much lower than the requirement. This temperature decrease, could reduce emissions by 69% compared to the emissions produced by HMA for heavy traffic.
WARM MIX ASPHALT FOR HEAVY TRAFFIC IN INDONESIA

Furqon AFFANDI; Neny KUSNIA NTI
INSTITUTE OF ROAD ENGINEERING
JI A.H Nasution 264 Bandung – Indonesia
e-mail: furqon_affandi@yahoo.com; neni.kusnianti@pusjatan.pu.go.id

Abstract

Indonesia has implemented the specification of hot mix asphalt (HMA) for heavy traffic, associated with increasing traffic load as well as high temperature and pavement in Indonesia, according to its location in equator. HMA for heavy traffic using modified asphalt requires mixing temperatures in AMP around 175-180 °C. This creates difficulties because of mixing temperature is higher than conventional HMA (asphalt pen 60) and will require more fuel and generate higher emissions. Warm Mix Asphalt (WMA) by adding some additive to asphalt 60 pen grade, can reduce mixing and compaction temperatures of about 40 °C (about 135 °C) compared to HMA for heavy traffic with quality equivalent to HMA using modified asphalt and meet specified requirements of mixture for heavy traffic. Rutting resistance of WMA is bigger around 1.89 times than HMA, which indicated by dynamic Stability of WMA and HMA is 5200 passing / mm and 2742 passing / mm respectively. Water resistance of WMA and HMA meet the requirements which greater than 80%. Resistance to material loss tested by Cantabro method (ASTMD 70 64) showed good results, where its material loss is only 2% and 3% respectively for WMA and HMA which much lower than the requirement. This temperature decrease, could reduce emissions by 69% compared to the emissions produced by HMA for heavy traffic.

Keywords: mixing temperature, Warm Mix Asphalt, additive, emission; rutting resistance.

1. INTRODUCTION

The length of road network in Indonesia 2008 reached 372.173km which includes 9.30% of national roads, 13.08% of provincial road, 77.43% of district / city roads and 0.18% of toll roads (Indonesia 2008). Approximately 97.12% of the paved roads in the national road is flexible pavement (Suhardi2013) while the rest is rigid pavement. In some national and provincial roads accommodate heavy traffic. Heavy traffic in Indonesia become one of the very important issue, because it requires special handling and maintenance of road sin order to be able to serve transportation of goods and people that support economic development.

Indonesia has made efforts to solve the problems on road pavement with heavy traffic load, i.e by setting specifications for heavy traffic, one of which is the use of modified asphalt for hot mix asphalt (HMA), so that the mixture with modified asphalt will be strong and resistant to the effects of heavy load compared to 60/70 pen bitumen, such as rutting. In addition to the heavy load, which must be considered is climate, Indonesia as a tropical country located on the equator has air temperature around 30°C throughout the year. Both factors greatly affect the performance of road pavement in Indonesia.

Hot mix asphalt using modified asphalt requires heating at Asphalt Mixing Plant (AMP) between 175-180°C, is much higher than using asphalt pen 60, which requires heating at about 160°C. Lower heating than those required, will result in damage to both the mixture and the pavement, because asphalt cannot properly mixed and compacted with aggregate, in practice, difficult to achieve the desired density resulting in voids in the mixture becomes large and oxidation occurred due to rain and the pavement will be damaged earlier than its design life.

On the other hand, high mixing temperature of the asphalt at AMP for modified asphalt leads to higher emissions and fuel use as well. This relates to the issue of global warming, environmental issues and fuel use which have become the world's attention. Indonesia is one country who are committed to it, aiming to reduce emissions by 20% by 2020.

Construction and maintenance of paved roads in Indonesia requires 1.3 million tons of asphalt per year (Sumadilaga, D.2007), and when the mixture has asphalt content of 6% by weight of the mixture, then the 1.3
million tons of asphalt will generate as much as 1.3 million tons/(0.06) = 21.6 million tons of hot mix asphalt. This of course will generate considerable emissions.

The problem of emission reduction in road pavement with maintaining the quality of road pavement is a challenge. Institute of Road Engineering (IRE) has conducted a laboratory study and immediately will carry out field study using additive to reduce emissions generated through the decrease of mixing temperature but still maintaining the asphalt mix quality.

2. LITERATURE STUDY

Hot Mix Asphalt for heavy traffic

Hot Mix Asphalt (HMA) is a type of pavement layers and commonly used in the world, including Indonesia. As the name implies, this mixture requires high temperature at the AMP to obtain good quality of mix and pavement. Mixing and compaction temperatures used in Indonesia today, based on the viscosity value of the asphalt between 170 ± 20 cSt for mixing and between 280 ± 30 cSt for compaction (Asphalt Institute 1993). Asphalt which commonly used in Indonesia for heavy traffic is modified bitumen with mixing temperature between 175-180 °C and compaction temperature of 160-165 °C.

The properties of HMA required for heavy traffic in Indonesia is shown in Table1, while the aggregate gradation and its properties shall meet the requirements as shown in Figure1 and Table2.

In anticipation of the effects of heavy loads on pavement performance, modified asphalt was used instead of using 60 pen bitumen grade. Significant difference of both requirements is the requirements of higher softening point with minimum of 54 °C, the requirement of elastic recovery properties, and Penetration Index for elastomer. Such requirements are needed to support the pavement to the effects of heavy traffic and temperature in Indonesia. Those parameters must be met in accordance with the type of asphalt mix, whether for the wearing course, binder course or base course

<table>
<thead>
<tr>
<th>Property</th>
<th>AC Mod</th>
<th>BC Mod</th>
<th>Base Mod</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effect of volumetric content (%)</td>
<td>Min.  43</td>
<td>40</td>
<td>35</td>
</tr>
<tr>
<td>Bitumen Content Absorption (%)</td>
<td>Max.  1.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number of Blow per Face</td>
<td></td>
<td></td>
<td>75 - 112</td>
</tr>
<tr>
<td>Void in Mix (%)</td>
<td>Min.  3.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Void in Mineral Aggregate (%)</td>
<td>Min.  15</td>
<td>14</td>
<td>13</td>
</tr>
<tr>
<td>Void filled with Bitumen (%)</td>
<td>Min.  63</td>
<td>63</td>
<td>60</td>
</tr>
<tr>
<td>Marshall Stability (kg)</td>
<td>Min.  1060</td>
<td>2250</td>
<td></td>
</tr>
<tr>
<td>Marshall Flow (mm)</td>
<td>Min.  3</td>
<td>4.5</td>
<td></td>
</tr>
<tr>
<td>Marshall Quotient (kg/mm)</td>
<td>Min.  300</td>
<td>350</td>
<td></td>
</tr>
<tr>
<td>Retained Marshall Stability after 24 hours soaking (%)</td>
<td>Min.  90</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dynamic Stability, passing/mm</td>
<td>Min.  2000</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Source: General Specification for Road and Bridge (Indonesia 2010)

Figure 1. Aggregate gradation of HMA for heavy traffic

One of specified parameters of asphalt mix for heavy traffic is rutting resistance stated by minimum dynamic stability of 2500.

Hot mix asphalt plus additive

In foreign countries, warm mix asphalt has been developed (Graham C. Hurley and Brian D Prowell), which can lower the mixing temperature to 30 °C lower than hot mix asphalt and can save fuel until 30% (Salem, and Fall, 2003).

In making asphalt mixture with lower temperature (Warm Mix Asphalt - WMA), a variety of additives have been produced with different technical approaches, such as chemical-based, wax based and water-based emulsion (Dong Woo Cho, 2011).
In WMA, the same result of coated aggregate and compaction can be obtained by using additive functioned as lubricants or viscosity decrease so it can be done at lower temperatures. The function of additive by utilizing its water content, can enlarge the volume of bitumen during mixing and easier in mixing and compaction (Hurley, GC. & Brian D. Prowell 2002).

In foreign countries, the use of fuel for WMA could reach about 32% lower than the fuel consumption for hot mix asphalt. In Korea, emission reduction from HMA with wax-based additive of 1% by weight of asphalt is 32%; 18%; 24% and 33% respectively for Carbon dioxide (CO2), Carbon monoxide (CO), sulfur dioxide (SOx) and Nitric Dioxide (NOx) (Cho, Woo Doong 2011; Cho, Woo Doong 2012).

Lower mixing temperature also means supporting a better environment. Another advantage of warm mix asphalt, in terms of technical factor is the oxidation in asphalt mix will be lower in line with the lower mixing temperature characterized by the softening point of the asphalt is slightly lower compared with that used in hot mix asphalt - HMA (Affandi 2013). Another advantage of this warm mix asphalt is to reduce thermal cracks.

3. RESULT AND ANALYSIS

The experiment of Warm Mix Asphalt

Analysis conducted on laboratory experiments, including tests of asphalt and designed asphalt mix properties such as mix design, volumetric properties, the strength of the mixture (Marshall Stability), water sensitivity, by comparing each of these properties between HMA and WMA mixture with additive compacted to the lowest temperature, but still meet the specifications defined in the 2010 General Specifications for Roads and Bridges, the Ministry of Public Works, Indonesia, 2nd revision. Asphalt mixture studied is asphalt mix for wearing course with 60 pen grade bitumen with additive, whereas aggregates used remains the same.

Material Test

The asphalt used is 60 pen bitumen grade, which meets the requirements according to the 2010 Indonesian general specifications of roads and bridges, as shown in Table2.

<table>
<thead>
<tr>
<th>No.</th>
<th>Property</th>
<th>Standard Test</th>
<th>Type I Asphalt Cement 60-70 Pen</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Penetration, 25°C (0.1mm)</td>
<td>SNI 06-2441-1991</td>
<td>50-70</td>
</tr>
<tr>
<td>2.</td>
<td>Viscosity at 135°C (cSt)</td>
<td>AASTHO T201-03</td>
<td>≥300</td>
</tr>
<tr>
<td>3.</td>
<td>Softening Point (°C)</td>
<td>SNI 06-2434-1991</td>
<td>≥48</td>
</tr>
<tr>
<td>4.</td>
<td>Penetration Index</td>
<td>-</td>
<td>≥1</td>
</tr>
<tr>
<td>5.</td>
<td>Ductility at 25°C (cm)</td>
<td>SNI 06-2434-1991</td>
<td>≥100</td>
</tr>
<tr>
<td>6.</td>
<td>Flash point, (°C)</td>
<td>SNI 06-2433-1991</td>
<td>≥232</td>
</tr>
<tr>
<td>7.</td>
<td>Solubility in Trichloroethylene (%)</td>
<td>AASTHO T44-03</td>
<td>≥99</td>
</tr>
<tr>
<td>8.</td>
<td>Specific Gravity</td>
<td>SNI 06-2441-1991</td>
<td>≥1</td>
</tr>
<tr>
<td>9.</td>
<td>Storage stability (°C)</td>
<td>ASTM D 5976 part 6.1</td>
<td>-</td>
</tr>
</tbody>
</table>

Residu test from TFOT (SNI-06-2440-1991) or RTFOT (SNI-03-6835-2002)

<table>
<thead>
<tr>
<th>No.</th>
<th>Property</th>
<th>Standard Test</th>
<th>Type I Asphalt Cement 60-70 Pen</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.</td>
<td>Loss of Heating (%)</td>
<td>SNI 06-2441-1991</td>
<td>≤0.8</td>
</tr>
<tr>
<td>11.</td>
<td>Penetration, 25°C (%)</td>
<td>SNI 06-2456-1991</td>
<td>≤4</td>
</tr>
<tr>
<td>12.</td>
<td>Penetration Index</td>
<td>-</td>
<td>≥1</td>
</tr>
<tr>
<td>13.</td>
<td>Elastic recovery (%)</td>
<td>AASTHO 301-88</td>
<td>-</td>
</tr>
<tr>
<td>14.</td>
<td>Ductility at 25°C (cm)</td>
<td>SNI 06-2432-1991</td>
<td>≥100</td>
</tr>
<tr>
<td>15.</td>
<td>Mineral passing No.100 sieve (%)</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

There are three kinds of aggregates used, i.e. coarse, medium and fine aggregates, taken from the same source, namely from the Sewo-Cirebon, West Java which quality meets the requirements according to the 2010 Indonesian General Specifications of Roads and Bridges. Aggregate gradation used in the mix design is gradation of wearing course specified in 2010 Indonesian general specifications.

Properties of 60 pen grade bitumen mixture with additive for WMA

Properties of 60 pen grade bitumen with additive is used in Warm mix asphalt for heavy traffic that could increase the softening point and could well compacted at lower temperatures or in a state called "warm". Mixing asphalt 60 pen grade with additive is done by using a special mixture at a temperature of 180°C for 1-1.5 hours with mixing speed of 5000-6000rpm.

The addition of additive increases the penetration about 54 dmm, softening point to about 61°C, still higher than the softening point requirement for modified bitumen i.e minimum 54°C, elastic recovery after Los On Heating (LOH) is 73 % much higher than the minimum requirements in modified asphalt by 60%.
Penetration Index of 60 pen grade bitumen plus additive for WMA both for fresh and after LOH process is higher than the requirements for modified asphalt for HMA heavy traffic, namely +1.26 and +0.87 respectively. The complete results of asphalt test with additive, the properties of 60 pen bitumen and modified asphalt requirements for Hot Mix Asphalt (HMA), which is used in Indonesia are shown in Table 3.

Table 3. The properties of 60 pen grade bitumen, 60 pen grade bitumen plus additive for WMA and modified asphalt requirements for heavy traffic.

<table>
<thead>
<tr>
<th>No.</th>
<th>Test</th>
<th>Method of Testing</th>
<th>Indonesian 60/70 pen grade</th>
<th>Indonesian 60/70 pen grade + Korean Additive + Sulfur for WMA (IRE)</th>
<th>Indonesian Modified Bitumen Specification (Elastomer)</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Penetration, 25°C, 100 g, 5 s</td>
<td>AASHTO T 49-03</td>
<td>65.3</td>
<td>54</td>
<td>min 40</td>
<td>dmm</td>
</tr>
<tr>
<td>2</td>
<td>Viscosity, 135°C</td>
<td>ASTM E 102 - 93</td>
<td>420</td>
<td>800</td>
<td>≤ 3000</td>
<td>cSt</td>
</tr>
<tr>
<td>3</td>
<td>Softening point</td>
<td>AASHTO T 53-8189</td>
<td>50.9</td>
<td>60.2</td>
<td>≥ 54</td>
<td>°C</td>
</tr>
<tr>
<td>4</td>
<td>Penetration Index</td>
<td>-</td>
<td>-0.331</td>
<td>+ 1.26</td>
<td>≥ 0.4</td>
<td>-</td>
</tr>
<tr>
<td>5</td>
<td>Ductility, 25°C, 5 cm / min</td>
<td>AASHTO T 51-89</td>
<td>&gt;140</td>
<td>&gt;140</td>
<td>≥100</td>
<td>Cm</td>
</tr>
<tr>
<td>6</td>
<td>Flash Point (COC)</td>
<td>AASHTO T 48-89</td>
<td>320</td>
<td>328</td>
<td>≥232</td>
<td>°C</td>
</tr>
<tr>
<td>7</td>
<td>Solubility in trichloroethylene (C2HCl)</td>
<td>ASTM D 2042-93</td>
<td>99.63</td>
<td>99.62</td>
<td>≥99</td>
<td>%</td>
</tr>
<tr>
<td>8</td>
<td>Specific gravity</td>
<td>AASHTO T 288-90</td>
<td>1.0359</td>
<td>1.0328</td>
<td>≥1</td>
<td>-</td>
</tr>
<tr>
<td>9</td>
<td>Loss on heating (TFOT)</td>
<td>ASTM D 1754</td>
<td>0.0174</td>
<td>0.0189</td>
<td>≤0.8</td>
<td>%</td>
</tr>
<tr>
<td>10</td>
<td>Penetration after LOH</td>
<td>AASHTO T 49-03</td>
<td>83.9</td>
<td>80</td>
<td>≥54</td>
<td>%</td>
</tr>
<tr>
<td>11</td>
<td>Softening point after LOH</td>
<td>AASHTO T 53-8189</td>
<td>53.4</td>
<td>61.05</td>
<td>-</td>
<td>°C</td>
</tr>
<tr>
<td>12</td>
<td>Penetration Index after LOH</td>
<td>-</td>
<td>-</td>
<td>+ 0.87</td>
<td>≥0.4</td>
<td>-</td>
</tr>
<tr>
<td>13</td>
<td>Ductility after LOH</td>
<td>AASHTO T 51-89</td>
<td>&gt;140</td>
<td>&gt;140</td>
<td>-</td>
<td>Cm</td>
</tr>
<tr>
<td>14</td>
<td>Separation, difference</td>
<td>ASTM D 5876 part 6.1</td>
<td>-</td>
<td>1,425</td>
<td>≤2.2</td>
<td>°C</td>
</tr>
<tr>
<td>15</td>
<td>Elastic Recovery (Fresh Bitumen)</td>
<td>AASHTO T 301-11</td>
<td>-</td>
<td>75,125</td>
<td>-</td>
<td>%</td>
</tr>
<tr>
<td>16</td>
<td>Elastic Recovery (After LOH)</td>
<td>-</td>
<td>73</td>
<td>-</td>
<td>≥60</td>
<td>%</td>
</tr>
</tbody>
</table>

From Table 3 above, it indicated that asphalt with additives improved its properties compared to 60 pen grade bitumen, even it is beyond the required properties in Indonesia modified bitumen specifications.

Asphalt mix using 60 pen grade bitumen plus additive (WMA)
As a comparison of asphalt mix with lower temperature (WMA) which will be tested, asphalt mix using modified asphalt (HMA) was also made in accordance with referred specifications, i.e. Marshall method (AASHTO T283-03-2004). Marshall test on asphalt content between 4.5% to 6.5% with an increase of 0.5% each, were carried out at mixing and compaction temperatures in accordance with modified asphalt test results that will be used.

Based on the test results of modified asphalt, obtained mixing temperature is between 175-180°C while compaction temperature is between 160-163°C. In this experiment, mixing and compaction temperature were determined to 175°C and 163°C respectively. Based on Marshall test results, obtained optimum bitumen content is 5.6%.

The experiment of Warm Mix Asphalt using 60 pen grade bitumen plus additive
The results of laboratory experiments that have been carried out, WMA properties with additive at mixing temperature of 135°C and compaction temperature of 123°C obtained optimum bitumen content 5.4%. From laboratory result warm mix asphalt using 60 pen grade bitumen plus additive, its volumetric properties and the strength of Marshall stability can meet the requirements specified for the asphalt mixture for heavy traffic as specified at 2010 Indonesian General Specifications of AC – Wearing Course for heavy traffic.
HMA and WMA Mixes properties at different temperature for mixing and compaction

To determine the ability of mix with additive, the mixture was tested at different mixing and compaction temperatures, at the optimum bitumen content according to the results obtained previously. Mixing / compaction temperature used is 175/163 °C; 165/153 °C; 155/143 °C; 145 / 133°C; 135/123 °C; 125/113 °C. For practical reason in presentation, only mixing temperature is included in horizontal axis in the figure, in fact, there is compaction temperature which follow (for example, written in the chart is 135 °C, it means mixing temperature is 135 °C and compaction temperature is 123 °C, and so on).

The effects of the decrease of mixing / compaction temperature on the mix properties were examined based on the Marshall test are shown in Figure 2 (a) to Figure 2 (e).

![Mixing / CompactingTemperature vs Density](image)

![Mixing / CompactingTemperature vs VIM](image)

![Mixing / CompactingTemperature vs VMA](image)

![Mixing / CompactingTemperature vs VFB](image)

![Mixing / CompactingTemperature vs Stability](image)

Figure 2 properties of WMA and HMA at different mixing and compaction temperatures

The density; From Figure 3 (a) the relationship between temperature and density, it showed that the declining trend is in line with the decrease in temperature. The density of WMA is always higher than HMA’s for the same mixing / compaction temperature as indicated by line of density of WMA which above than line of density of HMA. The decrease in density of the WMA is smaller than the decrease in HMA, this means that mixing / compaction temperature of the mixture plus additive is lower and would result higher density/compaction than the mixture with modified asphalt. As can be seen in that Figure, the
density of HMA which mixing and compacting at 175 °C/165°C is the same as the density of WMA which mixing and compacting at 135 °C / 123 °C.

**Voids in Mix - VIM:** the mixture with additive for WMA, VIM can be met at the mixing temperature of 133 °C and above, whereas the mixture with modified asphalt (HMA), VIM requirements can be met at a temperature of 163 °C and above, as shown in Figure 2 (b). This shows that the asphalt mix with additive (WMA) can be mixed at lower temperatures compared with the mixture use modified bitumen (HMA) associated with VIM fulfillment.

**Voids in Mineral Aggregate - VMA:** The effect of additive in the mixture on VMA, is shown in Figure 2 (c), where the range of mixing/compaction temperature between 125 °C to 175 °C, it showed that the VMA requirements can still be met by the mixture of modified asphalt or bitumen with additive for WMA as indicated by the line of VMA value of both mixture is above the minimum requirement of 15%.

**Void Filled Bitumen - VFB:** From Figure 2 (d), it is shown that asphalt mix plus additive (WMA), VFB can be met starting with mixing temperature of 130°C and above, where as modified asphalt mixture (HMA), VFB requirements can be met at the mixing temperature of 148°C and above. This figure reflects that in terms of the requirements of VFB, asphalt mix for WMA can be compacted at lower temperature than the temperature required by the mixture with modified asphalt (HMA).

**Stability:** stability requirements of asphalt mix with additive (WMA) can be met from mixing temperature of 125°C and above, whereas for the mixture with modified asphalt, stability can be met from the mixing temperature of 140°C and above, as shown in Figure 2 (e). From that Figure, it can be seen that stability of WMA is always higher that stability of HMA for the same mixing/compacting temperature.

**Rutting Resistance** was tested by means of Wheel Tracking Machine at a testing temperature of 60°C, wheel pressure of 6.4kg/cm2 to 1260 times the number of passes (Japan Road Association 1989). The results of testing on the asphalt mixture for WMA and the use of asphalt modification (HMA), shown in Figure 3.

![Graph of deformation resistance and dynamic stability tested using wheel tracking machine.](image)

From Figure 3, it is seen that deformation in mix plus additive for WMA is smaller than mixtures with modified asphalt, which is marked with the position of the graph is below the graph of mix with modified asphalt. Based on test results, the dynamic stability of asphalt mixture for WMA was obtained i.e. 5200 passes / mm, while the mixture with modified asphalt (HMA) is 2750 passes / mm as shown in Figure 3 (b), which means that the dynamic stability of asphalt mixture for WMA is bigger around 1.89 times. Obviously, rutting resistance of WMA is much higher than HMA with modified bitumen.

It should be kept in mind that the asphalt mix for WMA is mixed and compacted at a temperature of 135 °C / 123 °C while the mixture of HMA with modified asphalt is compacted at a higher temperature at 175 °C / 163 °C.

Furqon Affandi; Neni Kusniandi
The Resistance of mass loss (Cantabro test): To find out the resistance of asphalt mixture to material loss, testing was carried out using Cantabro method, where the specimen have been created the same as for Marshall test, it was inserted into the abrasion machine (Loss Angeles abrasion) and rotated up to 500 rounds. Every 50 rounds, the machine was stopped and weight loss of specimen was calculated, the same thing was done up to five hundred rounds. Test specimens with asphalt for WMA mixed and compacted at a temperature of 135°C/123°C lower than the temperature of the hot mix with modified asphalt at 175°C/163°C. The test results of the resistance to the material loss, is shown in Figure 4.

![Graph of material loss of asphalt mix for WMA and HMA](image1)

Figure 4. A graph of material loss of asphalt mix for WMA and HMA

The value of material loss was calculated at the rounds of 300, where the material loss of the mixture for WMA is 3 %, while HMA mixture with modified asphalt is 1.5 %. The value is small, still far below the maximum limit of 20% material loss ASTM 70 64 – (2005). The shape of the test specimens after Cantabro test both of WMA and HMA specimens are undamaged or unbroken.

Resistance to moisture induce damage, Indirect Tensile Strength (ITS): was carried out to find out the resistance to water effect in accordance with AASHTO T 283 -(2012), by comparing the indirect tensile strength of dry sample and after soaking. ITS of WMA relatively same as value of HMA mixture with modified asphalt i.e.86.3 % and 87% respectively. Both mixtures meet the requirements of water resistance expressed by ITS minimum value of requirements by 80%, as can be seen in Figure 5.

![Resistance to moisture induce damage](image2)

Figure 5. Resistance to moisture induce damage

The effect of mixing temperature on emission: From the experiment results above, asphalt mix with additive for WMA can be mixed at lower temperature at minimum of 135°C and even compaction temperature is much lower. Based on the study results conducted by Mallick, RB and Bergendah,J (2009), the CO2 emission (ppm) of HMA at temperature of mixing 175 °C is 1800 ppm, meanwhile WMA at temperature of mixing 135 C is 560 ppm. By using this study obtained emission reductions of WMA in this study is approximately by 69%.

4. CONCLUSIONS AND SUGGESTIONS

Conclusions
The conclusion that can be drawn from this experiment, is

1. WMA plus additive using 60 pen grade bitumen can lower mixing and compaction temperatures of 40 °C lower than HMA using modified bitumen
2. Technically, the quality of WMA is equivalent to hot mix such as stability, density, Voids in Mix, Void in Mineral Aggregate, Bitumen Void Filled
3. Rutting resistance of WMA is much more higher than rutting resistance of HMA use modified bitumen, approximately by around 1.89 times.
4. Resistance to Moisture –Induced Damaged of WMA and HMA is relatively the same and meet the requirement.
5. Resistance to the material loss of WMAs still far below the limit.
6. The decrease of mixing and compaction temperatures for WMA mixture plus additive can reduce emissions at AMP by 69 % than HMA.
7. The procedure of WMA design is no difference with the design of HMA.
Suggestion

1. Field trial of WMA is required in Indonesia

References


Cho, Doong Woo (2012); Development of Warm Mix Asphalt Technology ; Joint Work shop between KICT and IRE ; October 29 – 30 , 2012).


Development of Deduct Value Curves for Concrete Pavement based on Panel Rating Procedures

Young-Chan Suh1, Dae-Wook Park2, Vo Viet Hai3, Hong-Joon Kwon4

Abstract: The deduct values are importantly used to calculate the pavement condition index (PCI) for asphalt pavement or concrete pavement and the PCI is a key factor in a pavement management system. Korea pavement management authority has faced a problem that the PCI is good even though pavement is not in good condition; therefore, it is difficult to get maintenance funds from government. This study is to develop the deduct value curves which can represent current condition of existing pavement for the calculation of pavement condition index of concrete pavement instead of using existing deduct value curves developed from ASTM D 5340. To develop the deduct value curves of concrete pavement, definitions of severity for each distress type were made and then panel rating was conducted to decide the pavement condition based on pavement distress type, severity, and density. Results show that standard deviation of deduct values by panel rating is increased at higher severity level and as damage density increases. The deduct value curves based on panel rating could more adequately represent pavement condition of concrete pavement than by the existing deduct value curves (ASTM D 5340) in Korea.

1 INTRODUCTION

The original pavement condition index was developed in the mid-1960s to organize and coordinate the activities involved in achieving the best value possible for the available funds. In the development, the Portland Cement Concrete (PCC) pavement condition index was simplified by applying weighting values only to cracking, spalling, and faulting. In 1991–1992, a study was conducted to re-evaluate the pavement condition indices, and a new defect deduction scheme was proposed that modified deduct values for cracking, spalling at joints and cracks, faulting, pumping, patching, and scaling (Kay et al. 1993). At the time, few or no PCC pavement rehabilitation or reconstruction projects were planned for a foreseeable future, so the proposed procedure was not implemented.

According to Shahin (2005), the pavement network need to be managed and the pavement engineer’s experience tended to dictate the selection of Maintenance and Repair (M&R) techniques. The Pavement Management System (PMS) provided a systematic and appropriate method for selecting M&R and determining priorities and the optimal time of repair. Over 50 percent of repair cost and long periods of closure to traffic can be avoided if M&R is performed at the early stages of deterioration. Prediction of future pavement condition is not only essential for maintenance budget forecasting at the network level but also for determining the most cost-effective rehabilitation strategy at the project level (Bandara & Gunarthe 2001). Shahin et al. (1977) recommended that procedures for determining the deduct values of performing different M&R alternatives or areas should be developed to enable engineers to conduct meaningful, appropriate analysis to determine optimum alternatives. Broten and Sombre (2001) suggested that a slab by slab distress survey be conducted, where every occurrence of distress is identified, its severity noted, and its quantity measured. This information can then be converted into the standardized PCI values; however, it can also be used to determine maintenance quantities and estimate maintenance costs. Shahin et al. (1995) documented in a paper discussing the effect of sample unit size on the PCI calculation procedure, sample unit size should be kept within 40% of the recommended average size. Jackson (2009) found that the deduct values for PCC pavement currently used are basically those developed in the mid-1960s for the priority array program. They did not correlate to current understanding of PCC pavement deterioration stages. The approach taken in the development of the revised indices was kept purposefully simple. Pavement condition indices should not be more complicated and the

---

1 Professor, Dept. of Transportation Engineering, Hanyang University, 55 Hanyang Daehak Ro, Sangrok Gu, Ansang, Gyeonggi-Do, 426-791, Korea, E-mail: suhye@hanyang.ac.kr
2 (Corresponding Author) Associate Professor, Dept. of Civil Engineering, Kunsan National University, 558 Daehak Ro, Kunsan, Jellabuk-Do, 573-701, Korea, E-mail: dpark@kunsan.ac.kr
3 Ph.D Student, Dept. of Civil Engineering, Kunsan National University, 558 Daehak Ro, Kunsan, Jellabuk-Do, 573-701, Korea, E-mail: haiwo2310@gmail.com
4 Ph.D Student, Dept. of Transportation Engineering, Hanyang University, 55 Hanyang Daehak Ro, Sangrok Gu, Ansang, Gyeonggi-Do, 426-791, Korea, E-mail: agni83@nate.com
simpler the better. There are several advantages to keeping the system simple. The primary one is that it can be easily understood by all of the users of the PMS.

The main objective of this paper is to develop deduct value curves of concrete pavement to meet with pavement management condition in Korea. The deduct values of each distress type were rated by taking surveys based on pavement engineer’s experience. The rating results were analyzed and compared.

2 PAVEMENT CONDITION INDEX RATING SCALE

One pavement condition index (PCI) rating scale has been used for asphalt pavement and concrete pavement which can be used to decide what kind of maintenance should be used. In Korea, pavement maintenance agencies need different maintenance strategies for the asphalt pavement and concrete pavement. The PCI is a numerical index, ranging from 0 for a failed pavement to 100 for a perfect pavement condition. The PCI is calculated based on distress type, distress severity, and distress density. Recently, ASTM D 5340 (ASTM 2012) was changed PCI rating scale as more conservative and pavement description word was also changed to more skeptical. In Table 1, the PCI range for pavement condition and maintenance type is described for asphalt pavement and concrete pavement. Compared to ASTM D 5340, the developed PCI rating scale is changed to more conservative and pavement condition “Good” is restricted 91 – 100 while “Good” is 71 – 100 in ASTM D 5340. Different maintenance types are used to fix the problems of asphalt and concrete pavements. For example, overlay or overlay after grinding is recommended for asphalt pavement while overlay, overlay after grinding, or construction is recommended for concrete pavement in poor condition.

<table>
<thead>
<tr>
<th>PCI Range</th>
<th>Pavement Condition</th>
<th>Maintenance/ Rehabilitation (M&amp;R) Type</th>
<th>Concrete Pavement</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 – 91</td>
<td>Good</td>
<td>Routine maintenance</td>
<td>Routine maintenance</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Crack sealing</td>
<td>- Crack sealing</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Patching</td>
<td>- Joint replacement</td>
</tr>
<tr>
<td>90 – 81</td>
<td>Fair</td>
<td>Routine maintenance</td>
<td>Routine maintenance</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Crack sealing</td>
<td>- Crack sealing</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Patching</td>
<td>- Joint replacement</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Partial section fix</td>
</tr>
<tr>
<td>80 - 71</td>
<td>Mediocre</td>
<td>Preventive maintenance</td>
<td>Preventive maintenance</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Crack sealing</td>
<td>- Crack sealing</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Patching</td>
<td>- Joint replacement</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Partial Grind/ Overlay</td>
<td>- Partial section fix</td>
</tr>
<tr>
<td>70 – 61</td>
<td>Poor</td>
<td>Overlay and Grind/ Overlay (7.5 ~ 10 cm)</td>
<td>Asphalt overlay or Grind/ Overlay (over 10cm)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Construction</td>
</tr>
<tr>
<td>60 – 51</td>
<td>Very Poor</td>
<td>Grind/Overlay or Construction</td>
<td>Construction</td>
</tr>
<tr>
<td>50 - 0</td>
<td></td>
<td>Construction</td>
<td>Construction</td>
</tr>
</tbody>
</table>

3 DEFINITION OF SEVERITY LEVEL

To develop the deduct value curve of concrete pavement, definitions of severity level for various distress types of concrete pavement are needed. In our study, deduct value curves only for ten distress types out of sixteen distress types are developed because six distresses are rarely observed in concrete pavement of Korea. Table 2 shows the distress types which are developed deduct value curve and distress types which are used existing deduct value curves.

Table 2. Distress Type of Developed Deduct Value Curve

<table>
<thead>
<tr>
<th>Distress Type of Deduct Value Curve Developed</th>
<th>Distress Type of Existing Deduct Value Curve Used</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Linear cracks; Longitudinal, Transverse, and Diagonal Spalling (Longitudinal and Transverse Joint) Patching, Small Patching, Large and Utility Cuts Durability ("D") Cracking Scaling, Map Cracking, Shrinkage Corner Cracking Shattered Slab/ Intersecting Cracking Alkali Silica Reaction (ASR) damage

Severity level of distresses is changed and definition of severity level is modified as more concrete and quantitative. In Table 3, definitions of severity level of longitudinal and transverse cracking are summarized between ASTM D 5340 and proposed Korea definition. As shown in Table 3, the difference between two definitions is underlined. The proposed Korea definition is more conservative because the crack width which is more than 10 mm is rarely observed in Korea airports.

<table>
<thead>
<tr>
<th>Severity Level</th>
<th>ASTM D 5340</th>
<th>Korea</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>(1) Crack has little or minor spalling. (2) If nonfilled, it has a mean width less than approximately 3 mm (3) A filled crack can be of any width, but the filler material must be in satisfactory condition or the slab is divided into three pieces by low severity cracks</td>
<td>(1) Crack has little or minor spalling. (2) If nonfilled, it has a mean width less than approximately 3 mm (3) A filled crack can be of any width, but the filler material must be in satisfactory condition or the slab is divided into three pieces by low severity cracks</td>
</tr>
<tr>
<td>Medium</td>
<td>(1) Filled and nonfilled crack is moderately spalled. (2) A nonfilled crack has a mean width between 3 and 25 mm (3) A filled crack is not spalled or only lightly spalled, but the filler is</td>
<td>(1) Filled and nonfilled crack is moderately spalled. (2) A nonfilled crack has a mean width between 3 and 10 mm (3) A filled crack is not spalled or only lightly spalled, but the filler is</td>
</tr>
<tr>
<td>High</td>
<td>(1) filled or nonfilled crack is severely spalled (2) A nonfilled crack has a mean width greater than 25 mm (3) the slab is divided into three pieces by two or more cracks, one of which is at least high severity</td>
<td>(1) filled or nonfilled crack is severely spalled (2) A nonfilled crack has a mean width greater than 10 mm (3) the slab is divided into three pieces by two or more cracks, one of which is at least high severity</td>
</tr>
</tbody>
</table>

4 ANALYSIS OF PANEL RATING

Determining deduct values is the most difficult part. Ideally, the deduct values should be based on the measured impact that each pavement distress situation (type, severity, and density) has on pavement structural integrity and operational condition. However, measuring this effect requires extensive field testing. Due to the complexity of pavement systems and the current theoretical determination would also require a large research effort (Shahin et al. 1977). However, a subjective approach based on the collective estimation of experienced pavement engineers can be used to obtain and develop reasonable deduct values that can be used with confidence in every regions. The deduct values must first be determined based on existing knowledge, field tested and evaluated, and then revised where necessary.

Based on previous experience, an initial set of distresses was selected and severity levels were defined according to Tables 2 and 3. Practical photos of each distress type have been taken and collected in Korea airport. Preliminary surveys and discussion were then conducted with experienced professors and experts on field of pavement. A sample unit of 18 slabs was used in our study provided an adequate area of pavement for evaluation. An area of 18 slabs is large enough for meaningful distress measurement. The distress density was determined by counting the number of slabs in the sample unit that had a particular distress at a particular level of severity, and dividing it by the total number of slabs in the sample unit. Initial deduct values were determined based on the member’s experience and set according to the scale in Table 1 by subjectively
estimating the maximum deduct for each distress and severity level at a maximum density, and then assuming a
curvilinear relationship between the deduct value and distress density. Deduct value versus distress density
curves were derived in this manner for each distress type and level of severity.

The ratings were made for five levels of distress density. Each distress was rated at density levels of 1
slab cracked per 18 slabs, or 1/18, and 2/18, 4/18, 8/18, and 12/18. A plot of distress density versus mean deduct
value was developed, and a best fit smooth curve was fit through the points. The individual deduct values of
higher than about 20 percent than the mean were eliminated. This procedure was repeated for each of 8 distress
types and one to three levels of severity (low, medium, and high) of each type. The procedures were developed
and prepared for additional field testing and management.

Figure 1 illustrates the deduct value versus distress density curves for all 8 types of distress with 3
levels of distress severity; “H” is high severity, “M” is medium severity, and “L” is low severity. For
comparison, the dashed lines represent for the existing deduct value curves from ASTM D 5340 (ASTM 2012).
The standard deviation of deduct values by panel rating is increased at higher severity level and as damage
density increases. The curves were fit through the 5 points by power trendline. The general shape of the curves
is significant because it indicates the relative effect that amount or density of distress has on pavement condition.
The deduct values increase rapidly up to about 15 percent of distress density and then level off.

As compared to the existing deduct value curves, almost all of the new ones are lower at high severity
level but higher at low severity level and comparative at medium severity level except two. According to Figure
1(f), the new deduct value curves of Corner Crack are higher than the existing ones as distress density less than
30 percent, but lower when distress density greater than 30 percent. In Figure 1(g), the new deduct value curves
of Intersecting Crack are lower than the existing ones. In general, the gaps between the new curves at high,
medium and low severity level are closer than the existing ones.
Figure 1. Comparisons between Developed Deduct Value Curves and ASTM D 5360; (a) Linear cracks, (b) Spalling, (c) Patching, (d) D-crack, (e) Scaling, Map crack, Shrinkage, (f) Corner crack, (g) Intersecting crack, and (h) Alkali Silica Reaction (ASR)

Figure 2 illustrates the effect of different distress types on deduct values. Most of the curves have similar shapes, but their effects on the PCI differ greatly. For example, Intersecting cracks have a much higher deduct value than does Spalling. Besides, the curves alter in a different way with each severity level. As seen in Scaling, Map cracks, Shrinkage, the deduct value is low at low severity level but becomes the highest at high severity level.
5 VERIFICATION
The comparisons were conducted between deduct values by ASTM D 5340 and deduct values by developed in this study. Pavement images of Gimhae Airport were used to compare two different methods and investigation was conducted on 176 units. Figure 3 shows the relationships between PCI calculated by ASTM D 5340 and PCI calculated by developed deduct curves. As shown in Figure 3, the average PCI values of developed deduct curves are 8.9 lower than those by ASTM D 5340. As pavement condition become worse, the difference of PCI values between two methods become bigger.; therefore, authors believed that that developed deduct curves were more sensitive when pavement condition become worse. In some cases, low severity distresses by ASTM D
5340 were medium severity distresses newly developed distress definition.

![Figure 3. Comparisons of PCI between ASTM D 5340 and developed deduct curve](image)

6 CONCLUSIONS

In this study, a procedure was established for estimation of concrete pavement deducts value using engineer’s experience approach. The deduct value curves based on panel rating by pavement experts are more adequately represent pavement condition of concrete pavement in Korea than by the existing deduct value curves (ASTM D 5340). The comparisons between deduct curves of ASTM D 5340 and deduct curves by developed in this study showed that average PCI values were lower than PCI values by ASTM D 5340 and the lower PCI values indicated that the developed deduct curves can be used to represent concrete pavement condition in Korea.

ACKNOWLEDGEMENT

This paper was studied from a research project of “Development of Construction and Maintenance Technology for Low-Carbon Green Airport Pavements” funded by the Ministry of Land, Transport and Maritime Affairs (MLTM) and the Korea Institute of Construction & Transportation Technology Evaluation and Planning (KICTEP). The research project was conducted under the Center for Green Airport Pavement Technology (CGAPT) of Chung-Ang University. We are very grateful for their strong supports.

REFERENCES


<table>
<thead>
<tr>
<th>PAPER TITLE (90 Characters Max)</th>
<th>Asset Management to Drive Better Outcomes from Outsourced Road Maintenance and Renewal</th>
</tr>
</thead>
<tbody>
<tr>
<td>TRACK</td>
<td>Asset Management</td>
</tr>
<tr>
<td>AUTHOR (Capitalize Family Name)</td>
<td>POSITION                      ORGANIZATION                  COUNTRY</td>
</tr>
<tr>
<td>Wayne HATCHER</td>
<td>Principal Consultant Global Asset Management                        Opus International Consultants New Zealand</td>
</tr>
<tr>
<td>CO-AUTHOR(S) (Capitalize Family Name)</td>
<td>POSITION                      ORGANIZATION                  COUNTRY</td>
</tr>
<tr>
<td>Steven BROWNING</td>
<td>Transportation Asset Management Market Leader                        Opus International Consultants United Kingdom</td>
</tr>
<tr>
<td>E-MAIL (for correspondence)</td>
<td><a href="mailto:Wayne.hatcher@opus.co">Wayne.hatcher@opus.co</a></td>
</tr>
</tbody>
</table>

**KEYWORDS:**
Outsourcing, Asset Management, Procurement

**ABSTRACT:**
Outsourcing of road maintenance and renewal is becoming increasing widespread. Various outsourcing options are available to road authorities, many of which have been implemented internationally. The success of these outsourcing options though has been variable. This paper contests that the most successful of these contracts are strongly influenced by the adoption of asset management principals by the client agency and in turn through the contract to the Contractor. Where there is a strong adoption of Asset Management by an Agency resulting in clear articulation of the Authorities goals and objectives both internally and through the contract and specifications then it has been observed that there is an increased likelihood of success of the contract. Furthermore robust asset management, supported by excellent data management, can be used to better quantify and apportion risk which is known to result in better pricing from Contractors. This paper explores the linkage between Asset Management and outsourcing, including discussion on selecting appropriate contract models, data to support these models and risk apportionment.
Asset Management to Drive Better Outcomes from Outsourced Road Maintenance and Renewal

Wayne Hatcher¹, Steven Browning²

¹Opus International Consultants, New Zealand
Email for correspondence: wayne.hatcher@opus.co
²Opus International Consultants, United Kingdom
Email for correspondence: steven.browning@opus.co

1 INTRODUCTION

Internationally there are increasing trends towards outsourcing of road maintenance and renewal. In some countries, such as New Zealand and the United Kingdom, outsourcing has been the standard approach to undertaking works for more than 20 years. At the same time the discipline of Asset Management has been developing and therefore parallels between asset management and outsourcing can be drawn – specifically the needs of robust outsourcing and how these align with the principles of asset management.

Road Maintenance and often road renewal are activities that have been undertaken in-house by force account for many years. This has proven effective for many agencies and has become the “norm” in many parts of the world. In some countries though alternative methods of achieving similar outcomes have been implemented involving inputs from the private sector through outsourcing of road maintenance and renewal activity by term contract. These contracts can take many forms but essentially require a contractor to undertake a scope of work for a tendered sum.

The driver for agencies to consider alternative methods of delivery of road maintenance and renewal are wide and varied and may be as a result of several of the following factors including:

- Political pressure
- The need to reduce cost
- Risk transfer also to reduce cost
- Promoting organisational and workplace (cultural) change
- Seeking innovation through change
- Cost transparency through a contracted scope and price
- Reduced public sector employment
- Gaining greater control over timing and scope of work
- Private sector “reach-back” whereby the contractor has access to a wider range of skills than the public sector.

Simply choosing to outsource without first understanding key factors such as the respective roles of each party, the detailed extent of assets to be managed by the contractor and the equitable transfer of risk to the party best able to manage it can lead to unintended outcomes. Asset management, specifically the disciplines required for an effective asset management implementation provides the elements from which a robust outsourcing exercise can be undertaken. This increases the chance that the transition from force account, or even input contracting, to outsourcing involving increasingly complex delivery options such as performance based contracting will be successful.

2 OUTSOURCING

Road agencies have undertaken road maintenance, renewal and associated services by own forces, commonly referred to as ‘force account’, over a long period of time. Despite this history many agencies have increasingly been looking for ways to innovate, cut costs and gain efficiencies. Many of these agencies have engaged the private sector by adopting outsourcing procurement methods to undertake road maintenance, renewal and associated services on a term contract basis. Outsourcing of specialist road construction and maintenance activities has typically occurred as capability, economics and work volumes allow. Outsourcing of all road maintenance, often including consulting services and in some cases renewal, is relatively less common. There are though perceived benefits in adopting these approaches and various forms of procurement have been devised spanning a range of road maintenance services, procurement and renewal methods to achieve this. The relationship between the agency and the contractor has taken a number of forms mostly dependent upon the perceived skills of the contracting industry and the skills and knowledge of the agency.
Internationally several countries are recognized as leaders in outsourcing of road maintenance and renewal including United Kingdom, most Australian State Road Authorities, New Zealand, Argentina and several Canadian provinces. Contracts typically include unplanned reactive maintenance and emergency response and are typically term contracts for periods from three to 10 years. Outsourcing of renewal, whether rehabilitation and/or resurfacing, depends on the specialist nature of the work and the amount of perceived competition in the industry. Having more than one viable bidder is an essential element to ensure competitive pricing. Larger renewal work has typically been outsourced on a project-by-project basis or packaged into groups of smaller works. In most countries rehabilitation and renewal contracts usually take the form of annual contracts but there are several examples of longer periods being used such as three years for resurfacing contracts in some parts of New Zealand and “Category” contracts in England for the supply of materials, both to gain competitive pricing based upon volume and certainty of work. When considering contract periods beyond a single year having an understanding of future quantity and timing work requirements is essential. This knowledge can be gained though asset management.

It has been observed that as the contracting industry matures and as the agency gains a better understanding of its role, contract terms tend to increase towards the 10 –year horizon. One case, Western Australia, has adopted an evergreen approach in that the contractor’s performance is reviewed three-yearly and subject to satisfactory performance the contract can be extended a further three years – essentially forever although there may be a legal impediment to continuing “forever” within the state. Efficient management of the asset and its performance during the three year period are key factors in the decision to award an extension. This process is heavy underpinned by asset management.

The scope of services included within outsourced contracts appears to be aligned to historic practice and weather-related support such as snow clearance or flood response. In several instances where specialist equipment is involved, such as snow clearance equipment in England or specialist plant to relocate moveable motorway barriers in New Zealand, plant is owned and managed by the agency and operated by the contractor through the provisions of the term contract. The approach of the agency retaining ownership of some plant is deemed cost-effective as the plant is highly-specialized and typically lasts longer than the term of the contract. Where highly-specialised plant, not able to be transferred to another task, is required it can make the bidding process anti-competitive where one entity may have the plant (typically the incumbent) other entities are required to purchase plant and recover the cost through the contract. Therefore a prospective contractor would be severely disadvantaged by not owning or having access to the plant and hence ownership and often maintenance is retained by the agency. Operating the plant then becomes the responsibility of the contractor. Asset management has a key role to play here as it is through understanding asset life cycles that an agency will identify such issues.

Outsourcing of road maintenance and renewals though is not a trivial task. It requires a high degree of skill from both the agency to prepare the terms of reference appropriately and the contractor to execute those terms whilst at the same time achieving a reward/profit margin commensurate with the risk being taken. Asset management and the approaches required for its robust implementation are considered to be an excellent source of information to understand the:

- Roles required by the agency and contractor
- Extent of work required under the contract
- Risks best apportioned to each party
- Extent of data required to make informed decisions on future maintenance need
- Impact on pricing the above by the contractor.

3 ASSET MANAGEMENT FOR HIGHWAY INFRASTRUCTURE AND AGENCIES

Management of highway infrastructure has been at the forefront of the Asset management movement. For that reason many road agencies are well advanced on their highway asset management journey. Oftentimes though asset management is confused with maintenance management. Whilst maintenance management plays a critical role in the delivery of outcomes it does not fully encompass the range of asset management objectives. Furthermore, data and computing systems also play an important role in the delivery of outcomes but by themselves do not fully encompass asset management objectives.

A robust asset management implementation provides the agency with significant insight into:

- The purpose and objectives of the agency
- The extent of assets under its control including their location and quantity
When fully implemented, target service levels and measures against which performance can be assessed – linked to its purpose and objectives.

With this level of understanding an agency has the ability to objectively review its performance and identify prospective areas where efficiency gains might be achieved. Consequently this provides opportunity to assess the effectiveness of maintenance and renewal activity, reflect on how this activity is delivered and the outcomes achieved.

Management of highway maintenance and renewal is an integral part of the delivery of highway infrastructure. There are of course exceptions but the vast majority of highway authorities undertake investment in road maintenance and renewal as budget and skills permit. How the budget is derived and justified has a significant bearing on the activities that can be undertaken and hence the amount of maintenance and renewal that can be delivered. Traditional approaches of identifying maintenance need and determining the extent of financial provision, whilst having served for many years, are coming under increasing pressure. This pressure, brought about by financial constraints, a desire by politicians and taxpayers for greater transparency and an increasing need to consult widely, is driving different ways of viewing how agencies value maintenance thus the amount of money allocated to it.

The increasing need for transparency has become a significant aspect in the way funding authorities are seeking information in support of funding requests. This often means clear and concise arguments, backed by facts, detailing the need for investment with supporting information. This same approach is being adopted when seeking support for outsourcing options. The form of supporting information varies by agency but in principle it requires definition of the extent of assets to be maintained, a statement of their current performance and some form of commitment or forecast as to what will be achieved as a consequence of the investment. This sort of information allows the funding agency to weigh the relative benefits of investing in road maintenance and renewal as opposed to other funding pressures across government services.

The second pressure, the need to consult, is an increasing demand on roads agencies particularly in the western world, whereby key stakeholders and highway users are engaged to discuss proposed maintenance and renewal plans. The requirement is twofold: to inform and to seek feedback on direct and indirect impacts of proposed works. This information serves to connect the planning function with those directly affected by planned works and to seek wider input from stakeholders to identify aspects that may have been overlooked in the planning phase. This level of engagement is seen as critical as we move from managing assets to managing stakeholder expectations as budgets become increasingly constrained.

The provision of transparent information in support of funding requests and the need to consult can seem daunting challenges, especially in the absence of guidance and support. There are processes and procedures available to asset owners and agencies internationally that break the problem down into its constituent parts and provide guidance on the key aspects to be considered. Through the use of these it is possible to provide transparent and consistent information suitable for funding authorities and stakeholder in the form of asset management.

This very same information is required by the agency to justify its outsourcing options, understand its role and the prospective role of a contractor. This is essential to identifying a possible scope of work, risk transfers and the required skills of the agency and contractor to fulfill its obligations. With this information it may then be necessary to consult with prospective contractors to determine whether skills exist within the industry and gauge the extent of outsourcing that can be realistically implemented.

4. ASSET MANAGEMENT TO SUPPORT OUTSOURCING

Clearly defining scope is a crucial element of outsourcing. Asset management supports this by requiring an agency to understand its role in the provision of highway assets to support road function. This should be reflected in the functions of the agency and potential contractors. Having clear intent, defined by the agency, for a contractor to work within provides a framework to select appropriate skills and equipment to meet that intent and price appropriately. This is vital in establishing the overall outsourcing philosophy which, when implemented correctly, will translate to activities and interactions with stakeholders.

Furthermore the relative importance of the roles the contractor is required to fulfil should be evident and well-articulated in contract documentation as the agency will have reflected on its role in the provision of these functions and reflected in the terms of engagement. This usually raises key issues such as the level of support the agency requires in the event of severe weather or other events to assist with meeting its obligations to its road users and stakeholders.

Early attempts at outsourcing were often hampered throughout their duration by misrepresentation of the quantity of assets to be maintained. One of the main areas where this occurs is when the quantity of assets to be maintained has been under or over specified. Variations in the quantity of assets offers opportunities for contractors to
seek variations. Astute contractors will recognise these opportunities at the time of bidding and may, in certain circumstances, result in adjustments to rates to exploit the situation. A cornerstone of asset management is robust and accurate asset information – particularly the extent and quantity of assets within control of the agency. This same information provides the basis for quantifying the extent of assets to be maintained by the contractor. Therefore the more accurate the asset inventory data is the less opportunity exists for a contractor to exploit any weakness that may exist in the specified quantity to be maintained.

Figure 1 shows an overall asset management process as defined in ISO50000 Asset Management. Many of the elements required for asset management are very similar to those required for a robust outsourcing exercise particularly when considering higher order outsourcing models (discussed below). For example understanding who the stakeholders are and organisational context are important for communicating to contractors their role when providing services to the agency as they are usually the people that stakeholders and road users come in contact with the most. Asset management plans assist with understanding the extent of assets and the maintenance and renewal requirements, helping to identify the likely extent of maintenance and renewal required during the term of a contract. Asset management systems will usually provide the support elements to allow effective capturing of asset data and more importantly allowing the capture of works undertaken by the contractor for later analysis. Finally performance evaluation allows an agency to assess the effectiveness of works being undertaken and whether the contract model and/or the contractor is performing as expected.

5 CONTRACT MODELS

There are an increasing number of contract models available for procuring outsourced services. When considering the contract model to be employed the skills and capability of the agency and the contracting industry need to be considered. Outsourcing on a significant scale without first undertaking trials and at the same time upskilling agency managers and the contracting industry can lead to problems once the contract moves to the delivery phase. As the contract model moves from input through output and on to outcomes the skills and experience required to deliver and manage the contract change also as well as knowledge of the asset and the roles and responsibilities of the agency and contractor – all key parts of asset management.

Table 1: Spectrum of Contract Methods

<table>
<thead>
<tr>
<th></th>
<th>Input</th>
<th>Output</th>
<th>Outcome</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Common Name</strong></td>
<td>Force Account (Day Works)</td>
<td>Traditional</td>
<td>Performance Based</td>
</tr>
<tr>
<td><strong>Payment</strong></td>
<td>$/input</td>
<td>$/output</td>
<td>$/month subject to meeting performance standards</td>
</tr>
<tr>
<td><strong>Sophistication Required from Contractor</strong></td>
<td>Low</td>
<td>Medium</td>
<td>High</td>
</tr>
<tr>
<td><strong>Contractors’ Motivation</strong></td>
<td>As many inputs as possible, with no emphasis on efficiency.</td>
<td>As many outputs as possible – there is no</td>
<td>As little work as possible to deliver specified outcomes. Contractor seeks to be both</td>
</tr>
<tr>
<td>Input</td>
<td>Output</td>
<td>Outcome</td>
<td></td>
</tr>
<tr>
<td>-------</td>
<td>--------</td>
<td>---------</td>
<td></td>
</tr>
<tr>
<td>Client</td>
<td>Contractor</td>
<td>Contractor</td>
<td></td>
</tr>
<tr>
<td>Client</td>
<td>Client</td>
<td>Contractor</td>
<td></td>
</tr>
<tr>
<td>Low</td>
<td>Medium</td>
<td>High</td>
<td></td>
</tr>
</tbody>
</table>

It has been observed that agencies that have pursued asset management as an approach to managing their assets in advance of significant outsourcing are in a stronger position to transition to outcome contracts. They have a greater appreciation of what their road network requires, are able to articulate that in a contract and in communications to prospective contractors. Transport for Scotland for example has been a strong supporter of asset management and have been successfully outsourcing for a number of years. This contrasts with early outsourcing in Western Australia where Main Roads Western Australia were only at the early stages of asset management which contributed to a difficult outsourcing experience as control of key decisions, one the agency had a vested interest in in terms of its stakeholder relationships, were transferred to the contractor.

Data is the foundation of asset management and is also a core component that influences our ability to prepare and administer contracts effectively. As we move along the contract model continuum from force account, working under close direction, through output to performance-based contract models the need for accurate and robust asset and performance data increases. This is primarily due to the need to ensure value and more importantly that pricing reflects the contract requirements. Figure 2 below demonstrates this increasing data need.

![Figure 2 Data Requirements and the outsourcing model](image)

It should be noted that there is an increased cost to having good data (including accurate inventory and historical condition and achievement data) on the asset to be able to bid accurately and be aware of the risks. While this should happen with/without outsourcing (i.e. this is really a cost of establishing and maintaining good asset management and should occur irrespective of the drive for outsourcing), outsourcing often forces a higher level of investment in data. If not done well, data issues can lead to poor outcomes / disputes once the actual extent of the asset or the actual condition is better understood (often after the contract has started).

6. APPORTIONING RISK

Road agencies continually manage risk; either explicitly or implicitly. Risk takes many forms from political through to material supplies, design assumptions and workforce capability. Modern business practice and also asset management requires organisations to reflect, document risks the organisation is exposed to, quantify them and identify mitigation measures by which each risk can be minimised. Having a whole-of-organization view of risk often promotes an alternative view, from one of a provision of assets e.g. in the form of roads and bridges, to one of managing and mitigating risk e.g. in the form of meeting political aspirations and those of road users. The level of risk an agency is
prepared to accept is often influenced by the legislative framework the agency operates within and the case law that may have developed around any legislation that may be in place. This is further influenced by the level of funding the agency has available to meet its obligations and mitigate its identified risks.

Having an informed view of risk, and more particularly the mitigation measures identified to minimise that risk, is important to knowing whether the road agency will be able to meet its obligations. Mitigation measures are therefore a critical element and by achieving them the agency stands a good chance of meeting its obligations. Therefore whoever has ownership of the mitigation measure also has responsibility for ensuring its successful execution or identifying if further corrective action is required. Therefore identifying who has the ability to influence and mitigate risk becomes an important process when considering outsourcing options. This is vitally important as inequitable transfer of risk to the contractor can result in substantial risk-pricing or considerable variability in prices received from bidders.

Early outsourcing contracts often took the opportunity to transfer as much risk as possible to the contractor in the attempt to alleviate the agency of is apparent obligations. The reality was that the agency was ultimately still responsible for the risk, if not directly then indirectly, as the receiver of the benefit of the actions or inaction of the contractor; therefore never fully alleviating themselves of all risk. Furthermore contractors are typically private sector commercial bodies that exist to return a dividend to their shareholders. Therefore they will price to recover risk and hence it is important to be aware of the risks being transferred to ensure equitable pricing occurs and to avoid paying for the same risk twice – once through the contract and secondly as the Principal to the contract should the contractor default.

As outsourcing models move from input to outcomes risk transfer occurs; transferring risk from the agency to the contractor. For these high-end contract models where a reasonable amount of risk transfer occurs the extent of risk pricing can start to dominate a detailed bottom-up price as demonstrated in Figure 3. In reality both approaches are usually adopted when pricing with both outcomes compared to determine the bid sum.

![Figure 3 Contracting Models – Determining Factors](image)

6 CONCLUSIONS

Outsourcing by asset owners as a method of delivering road maintenance and renewal has been undertaken over at least the last 20 years. During that time asset management, as a discipline, has emerged and is now fully supported by the International Organization of Standardization as ISO55000.

The adoption of the principles of asset management, without needing to achieve ISO55000 certification, provides significant building blocks for an asset owner to assess its role, the extent of assets under its control and the standards at which those assets are to be delivered to meet stakeholder and organizational aspirations. This process provides valuable insight into what options are available for an asset owner when considering outsourcing and more specifically allows for a better articulation of scope and if necessary the outcomes and performance measures to be achieved by a contractor. Furthermore, and most importantly, a robust implementation of asset management will include a critical review of the risks being managed by the agency and therefore a clear view of which risks are best transferred to the contractor. This requires an understanding of who is best to manage and own a specific risk and therefore who should be responsible for its financial implications.

A key support element to outsourcing is data. Having data quantifying the extent of the asset allows the contractor to reliably price for the delivery of services and to reduce its risk on uncertainty relating to quantity. For the
agency this often ensures pricing is relatively robust and should mitigate price variation as a consequence of quantity uncertainty.

The building blocks of asset management are highly aligned to the requirements of establishing a robust outsourced contract model. It is therefore recommended that before asset owners consider outsourcing as an option they review the asset management building blocks to assist with preparing clear and transparent contract documents. This will contribute greatly to achieving the goals and objectives of the agency through outsourced service delivery.

7 ACKNOWLEDGEMENTS

The authors wish to acknowledge Mr. Tony Porter of Opus International Consultants for his support, allowing us to prepare and submit this paper. His guidance and mentorship has been invaluable.

8 CITATIONS AND REFERENCES


Quantifying Risk for Safer Roads Using ChinaRAP - Case Study of S102 Trunk Road in Shaanxi Mountain Road Demonstration Project

KEYWORDS:
ChinaRAP, countermeasures, road safety assessment, Star Ratings and Safer Roads Investment Plans

ABSTRACT:
The research introduces a ChinaRAP assessment of a 60.3km long trunk road in Shaanxi province, within a road safety improvement demonstration project. The assessment quantified infrastructure-related risk by giving the road risk scores and ratings for each road user type in a systematic way by coding the road attributes in 100 meter segments of road. Moreover, economic analysis to the project with certain budget limits were applied which would help the executives to make decisions on the priorities for improving the road and also helped the local designers to understand quantitatively the safety values of the countermeasures available to them.
Quantifying Risk for Safer Roads Using ChinaRAP - Case Study of S102 Trunk Road in Shaanxi Mountain Demonstration Project

Han Hu¹
Xiaohong Ma²
Greg Smith³
Tiejun Zhang¹

¹Research Institute of Highway, China
²Shaanxi Department of Transport, China
³IRAP

Email for correspondence: h.hu@rio.cn

1 INTRODUCTION

Road crashes are one of the top three causes of death for people aged between 5 and 44 years of age worldwide. According to the World Health Organisation, 1.24 million people were killed on the world’s roads in 2010. This is unacceptably high. Road traffic injuries take an enormous toll on individuals and communities as well as on national economies. Middle-income countries, which are motorizing rapidly, are the hardest hit. (WHO, 2013).

In order to help prevent road deaths and injuries, the government of Shaanxi and the Asian Development Bank (ADB) are developing the Shaanxi Mountain Road Safety Demonstration Project. The project focuses on some 800km of roads in An kang and Shangluo, which together account for about 15% of the province’s population. According to the project Concept Paper, “Lack of adequate transport accessibility and highly unsafe road conditions are major constraints on the social and economic development of the region” (ADB, 2013).

The government has recognized the urgent need for action on road safety, and this is reflected by the fact that the People’s Republic of China (PRC) is a signatory of the United Nations ‘Decade of Action for Road Safety, 2011-2020’. Further, in 2011, the State Council of the PRC issued the ‘Safety Plan in the 12th Five-Year Plan’ which establishes quantitative planning objectives for road safety at the national level. The project is also aligned with the ADB’s Country Partnership Strategy, 2011-2015 for the PRC in the areas of safety and social sustainability (ADB, 2013).

The ChinaRAP team at the Research Institute of Highway (RIOH), Ministry of Transport, was engaged to lead the road safety component of the project preparation phase. ChinaRAP is led by RIOH in partnership with the International Road Assessment Programme (iRAP). The project is in part designed to demonstrate the impact of rigorous and data driven road safety assessment and design practices to reduce crash rates and provide a model that can be replicated in other provinces as well as in other developing countries.

This paper focuses on one of the trunk roads included in the project: state highway S102. It describes the ChinaRAP assessments of the existing road and the preliminary design, and the proposed additional improving opportunities for local authorities and designers in a systematic manner.

¹ This project was formerly known as the Shaanxi Trunk Road Improvement Project.
2 STUDY METHODOLOGY

For safe road infrastructure improvement, there are many guiding methods, such as safety audit, accident analysis, safety assessment and black spot identification. The method of ChinaRAP safety assessment involves Star Ratings and Safer Roads Investment Plans which based on the systematic road safety engineering (RIOH, 2014). The method draws the iRAP methodology and techniques developed by RIOH. More information about the iRAP (ChinaRAP) methodology is available at http://www.irap.org/en/about-irap-3/methodology.

ChinaRAP Star Ratings involve an inspection of road infrastructure attributes and the traffic operation condition that are known to have an impact on the likelihood of a crash and its severity. The method involves recording more than 50 road attributes at 100 metre intervals along a road. It is a tool to measure the safety level in an easy and objective way on the basis of the careful investigation on actual basic data of roads is provided in one project, and five star rating is the safest and one star rating is the most dangerous.

Safer Roads Investment Plans (SRIP) provide an optimized strategy for safety improvement. A SRIP is a prioritized list of countermeasures that can cost effectively improved Star Ratings and reduce infrastructure-related risk. The plans are based on an economic analysis of a range of countermeasures, which is undertaken by comparing the cost of implementing the countermeasure with the reduction in crash costs that would result from its implementation. The plans contain extensive planning and engineering information such as road attribute records, countermeasure proposals and economic assessments for 100 meter segments of a road network. This could also provide reliable reference for the safety management policy making and prediction in the future.

The application of the system helped to guide executives of Shaanxi province in developing an understanding of the risk distribution on their road network for different road users and adopt specific safety measures on high risk sections. The system can also compare and analyze the operation risks at different time for the road network of commercial vehicles, effectively estimate the effect of the safety measures, and provides reference for the safety management policy making and prediction in the future. The system is divided into three major modules, including rapid collection of traffic safety information, road network risk assessment and economic and effect analysis.

With a safety goal of lifting all roads to at least 3-stars, to date the follow steps have been taken:

- Road surveys were undertaken on the existing roads (2012).
- Preliminary ChinaRAP results were produced and provided to the Government of Shaanxi.
- RIOH and iRAP staff met with Shaanxi Government Executing Agency (EA) and local design staff to discuss safety on S102 (May 2014). This included: instructions on how to interpret ‘Strip Plans’ (which list safety countermeasure suggestions for each 100 metre segment of road); instructions on how to use the ChinaRAP online ‘Demonstrator’ to test the effect of different design options on Star Ratings; illustrated the online Road Safety Toolkit; and discussion on the Ministry of Transport Highway Safety Enhancement Project guidelines.
- Suggestions for safety countermeasures for each 100 metre segment of road were provided to local designers for consideration (May 2014).
- Preliminary designs were produced by local designers and received by RIOH and iRAP staff, and assessments undertaken (April-June 2014).

3 ROAD INSPECTION DATA

S102 is a north-south provincial trunk road which connects two counties at Ankang City. The existing road is Class III standard which will be Class II standard road with safety concerns to be considered. The total length of the road that was assessed is 67.3km/h.

Table 1 lists a sample of key road attributes that were recorded along the road at both the time of the baseline
inspection (2012) and those that have been recorded to reflect the preliminary design. The ‘road inspection’ of the preliminary designs involved recording data from the design drawings and schedules. Table 1 allows for a direct comparison of the road attributes and helps to illustrate the changes that were proposed by local designers in the preliminary designs. It is to be noted that: the existing design had 2.3 km of trees greater than 4 in. in diameter. In Table 1, The preliminary design now has 11.4 km of hazardous trees. This is because that for the designs, if the situation at roadside is not fully described, the coder will code the roadside as the most dangerous section: Tree >=10cm. The key features of the preliminary design that relate to risk of death and serious injury are:

- realignments which reduce the overall length of the road and reduce the number and sharpness of curves
- installation of new safety barriers, including replacing old safety barriers
- increasing lane width, adding some paved shoulders and improving pavement condition
- improving delineation
- an increase in operating speeds.

**TABLE 1 Changes of road attributes in each design stage**

<table>
<thead>
<tr>
<th>Road Attribute / Category</th>
<th>Existing (km)</th>
<th>Preliminary Design (km)</th>
<th>Difference (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Length</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>67.3</td>
<td>60.3</td>
<td>-7</td>
</tr>
<tr>
<td><strong>Roadside severity - passenger side object</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tree &gt;=10cm</td>
<td>2.3</td>
<td>11.4</td>
<td>9.1</td>
</tr>
<tr>
<td>Non-frangible sign/ post./pole &gt;=10cm</td>
<td>7</td>
<td>5.5</td>
<td>-1.5</td>
</tr>
<tr>
<td>Unprotected safety barrier end</td>
<td>0</td>
<td>16.8</td>
<td>16.8</td>
</tr>
<tr>
<td>Aggressive vertical face</td>
<td>19.8</td>
<td>2.1</td>
<td>-17.7</td>
</tr>
<tr>
<td>Upwards slope - (15° to 75°)</td>
<td>1.3</td>
<td>0</td>
<td>-1.3</td>
</tr>
<tr>
<td>Deep drainage ditch</td>
<td>18.4</td>
<td>2.8</td>
<td>-15.6</td>
</tr>
<tr>
<td>Large boulders &gt;=20cm high</td>
<td>2.3</td>
<td>1.5</td>
<td>-0.8</td>
</tr>
<tr>
<td>Non-frangible structure/bridge or building</td>
<td>10.3</td>
<td>2.1</td>
<td>-8.2</td>
</tr>
<tr>
<td>Frangible structure or building</td>
<td>3.6</td>
<td>0.5</td>
<td>-3.1</td>
</tr>
<tr>
<td>Safety barrier - concrete</td>
<td>0</td>
<td>17.2</td>
<td>17.2</td>
</tr>
<tr>
<td>Upwards slope - (&gt;= 75°)</td>
<td>2</td>
<td>0.3</td>
<td>-1.7</td>
</tr>
<tr>
<td>No object</td>
<td>0.3</td>
<td>0.1</td>
<td>-0.2</td>
</tr>
<tr>
<td><strong>Paved shoulder</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>None</td>
<td>0.2</td>
<td>0</td>
<td>-0.2</td>
</tr>
<tr>
<td>Narrow (≥ 0m to &lt; 1.0m)</td>
<td>67.1</td>
<td>10.9</td>
<td>-56.2</td>
</tr>
<tr>
<td>Medium (≥ 1.0m to &lt; 2.4m)</td>
<td>0</td>
<td>49.4</td>
<td>49.4</td>
</tr>
<tr>
<td>Wide (≥ 2.4m)</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td><strong>Intersection quality</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Poor</td>
<td>0.9</td>
<td>0.6</td>
<td>-0.3</td>
</tr>
<tr>
<td>Adequate</td>
<td>0.5</td>
<td>0.8</td>
<td>0.3</td>
</tr>
<tr>
<td>Not applicable</td>
<td>65.9</td>
<td>58.9</td>
<td>-7</td>
</tr>
<tr>
<td><strong>Number of lanes</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>One</td>
<td>67.3</td>
<td>60.3</td>
<td>-7</td>
</tr>
<tr>
<td><strong>Lane width</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Narrow (≥ 0m to &lt; 2.75m)</td>
<td>0.9</td>
<td>0</td>
<td>-0.9</td>
</tr>
<tr>
<td>Medium (≥ 2.75m to &lt; 3.25m)</td>
<td>63.4</td>
<td>0</td>
<td>-63.4</td>
</tr>
</tbody>
</table>
### 4 STAR RATINGS

Table 2 displays the Star Ratings for the existing road, preliminary design and preliminary design with suggested ‘extra safety package’ of countermeasures. The package of suggested countermeasures is based on a budget of approximately CNY 41 million (more information about the package is provided below).

<table>
<thead>
<tr>
<th>Road Attribute / Category</th>
<th>Existing (km)</th>
<th>Preliminary Design (km)</th>
<th>Difference (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Curvature</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Very sharp</td>
<td>10.9</td>
<td>4.9</td>
<td>-6</td>
</tr>
<tr>
<td>Sharp</td>
<td>18</td>
<td>17.1</td>
<td>-0.9</td>
</tr>
<tr>
<td>Moderate</td>
<td>16.2</td>
<td>15.1</td>
<td>-1.1</td>
</tr>
<tr>
<td>Straight or gently curving</td>
<td>22.2</td>
<td>23.2</td>
<td>1</td>
</tr>
<tr>
<td><strong>Delineation</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Poor</td>
<td>21.3</td>
<td>0</td>
<td>-21.3</td>
</tr>
<tr>
<td>Adequate</td>
<td>46</td>
<td>60.3</td>
<td>14.3</td>
</tr>
<tr>
<td><strong>Street lighting</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Not present</td>
<td>55.7</td>
<td>50.7</td>
<td>-5</td>
</tr>
<tr>
<td>Present</td>
<td>11.6</td>
<td>9.6</td>
<td>-2</td>
</tr>
<tr>
<td><strong>Pedestrian crossing</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No facility</td>
<td>67.3</td>
<td>60.3</td>
<td>-7</td>
</tr>
<tr>
<td><strong>Operating Speed (85th percentile)</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>60km/h</td>
<td>0</td>
<td>60.3</td>
<td>60.3</td>
</tr>
<tr>
<td>50km/h</td>
<td>67.3</td>
<td>0</td>
<td>-67.3</td>
</tr>
</tbody>
</table>

Table 2. Star Ratings Comparison

* Sections of road that are under construction or do not have that road user present are not given a rating and so recorded as
not applicable (N/A).

The analysis shows that the preliminary designs would result in improvement in Star Ratings for vehicle occupants and marginal improvements for motorcyclists, though the Star Ratings for pedestrians and bicyclists would get worse. However, if the extra safety package were applied, the Star Ratings would increase further for vehicle occupants and motorists and virtually all the sections of road with pedestrian or bicyclist flows would achieve at least 3-stars for those road users.

Figures 1 and 2 illustrate new realignments proposed in the preliminary designs and the vehicle occupant Star Ratings. Figure 1 shows that while the existing alignment is rated 2-stars, the new alignment would achieve 3-stars. Figure 2 shows that while the existing alignment is rated 2-stars, the new alignment would achieve 4-stars.

Figure 1. Vehicle occupant Star Ratings for an existing section and proposed realignment
Figure 2. Vehicle occupant Star Ratings for an existing section and proposed realignment

5 EXTRA SAFETY PACKAGE

Following the production of Star Ratings, an economically viable ‘extra safety package’ that could largely achieve the goal of ensuring that the road achieves at least 3-stars for each road user and was within budget. For this analysis, a budget of CNY 41 million was set, although it was anticipated that local designers will review the list of countermeasure and make the final decision about what countermeasures could be adopted. Table 3 lists some of the most promising countermeasures.

Table 3. Examples of countermeasures in the extra safety package (20 years)

<table>
<thead>
<tr>
<th>Row Labels</th>
<th>Length (km)</th>
<th>Fatalities and serious injuries that could be prevented</th>
<th>Present value economic benefits</th>
<th>Present value cost</th>
<th>Benefit cost ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roadside barriers - driver side</td>
<td>20.3</td>
<td>298</td>
<td>CNY 106 501 502</td>
<td>CNY 9 885 000</td>
<td>11</td>
</tr>
<tr>
<td>Improve curve delineation</td>
<td>24.4</td>
<td>257</td>
<td>CNY 92 058 223</td>
<td>CNY 3 204 497</td>
<td>31</td>
</tr>
<tr>
<td>Traffic calming</td>
<td>23.9</td>
<td>242</td>
<td>CNY 86 675 601</td>
<td>CNY 631 903</td>
<td>137</td>
</tr>
<tr>
<td>Roadside barriers - passenger side</td>
<td>14.1</td>
<td>187</td>
<td>CNY 66 932 334</td>
<td>CNY 6 895 000</td>
<td>10</td>
</tr>
<tr>
<td>Central hatching</td>
<td>13.3</td>
<td>162</td>
<td>CNY 58 092 996</td>
<td>CNY 5 122 957</td>
<td>12</td>
</tr>
<tr>
<td>Roundabout</td>
<td>0.5</td>
<td>111</td>
<td>CNY 39 575 982</td>
<td>CNY 4 772 954</td>
<td>8</td>
</tr>
<tr>
<td>Unsignalised crossing</td>
<td>4.8</td>
<td>102</td>
<td>CNY 36 563 189</td>
<td>CNY 3 767 624</td>
<td>10</td>
</tr>
</tbody>
</table>

6 ECONOMIC ANALYSIS

The creation of the extra safety package involved an economic analysis of the safety components of the preliminary design and possible additional countermeasures. Table 4 lists the supporting data.

The following tables summarize the numbers of deaths and serious injuries, and crash costs and benefits for the existing road, preliminary design and preliminary design with the suggested extra safety package.

Table 4. Supporting Data

<table>
<thead>
<tr>
<th>Variable</th>
<th>Value</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Start year</td>
<td>2016</td>
<td></td>
</tr>
<tr>
<td>Analysis period (years)</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>Discount rate</td>
<td>12%</td>
<td>Asian Development Bank</td>
</tr>
<tr>
<td>GDP per capita (current price)</td>
<td>CNY 45 911</td>
<td>International Monetary Fund, World Economic Outlook Database, 2014</td>
</tr>
<tr>
<td>Value of life (million)</td>
<td>CNY 3.01</td>
<td>Ministry of Transport</td>
</tr>
<tr>
<td>Value of life / GDP per capita ratio</td>
<td>66</td>
<td></td>
</tr>
</tbody>
</table>
The economic analysis showed that the preliminary designs developed by local designers would result in an overall reduction in fatalities and serious injuries. It is estimated that, other things being equal, the design changes would result in 510 (6%) fewer fatalities and serious injuries over 20 years, saving CNY 182 million in crash costs avoided. The ‘extra safety package’ is estimated would result would result in 2,327 (29%) fewer fatalities and serious injuries, saving CNY 832 million in crash costs avoided.

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Fatalities and serious injuries</th>
<th>Fatalities and serious injuries saved (%)</th>
<th>Economic benefit (m)</th>
<th>Economic cost (m)</th>
<th>Benefit cost ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Existing</td>
<td>7925</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Preliminary design</td>
<td>7415</td>
<td>510</td>
<td>6%</td>
<td>CNY 182</td>
<td>CNY 1089</td>
</tr>
<tr>
<td>Preliminary design + extra safety package</td>
<td>5598</td>
<td>1817</td>
<td>25%</td>
<td>CNY 650</td>
<td>CNY 41</td>
</tr>
<tr>
<td>Total</td>
<td>5598</td>
<td>2327</td>
<td>29%</td>
<td>CNY 832</td>
<td>CNY 1130</td>
</tr>
</tbody>
</table>

**7 CONCLUSIONS**

The Star Ratings and SRIP for S102 on existing, preliminary design and extra safety package showed that:

(1) The star ratings are showing some improvements for the preliminary designs. But pedestrians ratings in particular are still low.

The preliminary designs developed by local designers would result in an increase in the length of road rated 3-stars or better for vehicle occupants and motorcyclists, but a reduction for pedestrians and bicyclists. This would translate into an overall reduction in fatalities and serious injuries. It is estimated that, other things being equal, the design changes would result in 510 (6%) fewer fatalities and serious injuries over 20 years, saving CNY 182 million in crash costs avoided.

(2) The proposed ‘extra safety package’ is estimated to cost CNY 41 million and would result in the road almost all achieving at least 3-stars for all road users. If the extra safety package were added to the designs, it would result in 2,327 (29%) fewer fatalities and serious injuries, saving CNY 832 million in crash costs avoided. This is to further improve the safety benefits of the road upgrades.

For the S102 trunk road, we identified many opportunities to improve the preliminary designs. The adoption of these will be subject to decisions by the EA. Overall the systematic assessment of risk on the roads, and the interactive design with the local designers, is helping to create a safer outcome which can be objectively measured.
For future works, the researchers proposed that:

- The local designers shall read the ‘Strip plans’ provided by the researchers. Each countermeasure on the plan should be regarded as a suggestion and must be subject to additional consideration during detailed design.

- The researchers will reassess the detailed design based on the feedback on ‘strip plan’ for preliminary design from the designers to see how the designs will be changed by the risk assessment.

- Prioritized countermeasures might need to be addressed to some of the roads for safety improvement and other means of safety improvements such as capacity building other than infrastructure improvement shall be considered.

8 ACKNOWLEDGMENTS

The Shaanxi Mountain Road Safety Demonstration Project is an initiative of the Government of Shaanxi and the Asian Development Bank. RIOH and iRAP are engaged under contract to the ADB. The project is also supported by the NSF Program of China (No. 51308263).

9 REFERENCES


World Health Organization, 2013, Global Status Report on Road Safety: Supporting A Decade of Action

RIOH, iRAP, 2014, Shaanxi Mountain Road Safety Demonstration Project, Road Safety Inception Report, ADB Project Number 46042.

RIOH, iRAP, 2014, Shaanxi Mountain Road Safety Demonstration Project, Interim Report part 4, Assessment of Trunk Road S102 Preliminary Design, ADB Project Number 46042.
# PAPER TITLE
CONTINUOUSLY REINFORCED CONCRETE PAVEMENT (CRCP) OVERLAY CONSTRUCTION AS A SOLUTION FOR CONCRETE PAVEMENT DETERIORATION REHABILITATION IN TANGERANG-MERAK TOLL ROAD

## TRACK

## AUTHOR
(Capitalize Family Name)

<table>
<thead>
<tr>
<th>MAINTENANCE DIVISION HEAD</th>
<th>PT MARGA MANDALASAKTI</th>
<th>INDONESIA</th>
</tr>
</thead>
<tbody>
<tr>
<td>AGUNG PRASETYO</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

## E-MAIL
(for correspondence)
goenx2000@yahoo.co.id, sigit.hermanto@margamandala.co.id

## KEYWORDS:
Method, CRCP, pavement damage, Tangerang - Merak Toll Road

## ABSTRACT:

Tangerang – Merak Toll Road with 72.45 km length was constructed between the years 1990-1996. The highways connect the city of Jakarta and the ferry port of Merak and simultaneously connecting Java to Sumatra island. The construction of the pavement on the highway consists of two types of pavements, i.e. rigid pavement and flexible pavement. It was planned the service lifetime for rigid pavement for 20 years, but due to the escalation of overloaded trucks is very excessive, the service lifetime reduce significantly. It is characterized by the onset of many damages, such as pumping, cracks in slab (block, edge, longitudinal and transverse cracks) etc. In the year of 2003 Institute of Road Engineering Agency for Research and Development of Ministry of Public Works carried out some investigation in this Toll Road and the results were as follows: The service lifetime for the first lane of the highway toward to Jakarta has been exceeded in the year 2001, while the second lane in the year 2003. Reversely for the direction to Merak, the service lifetime for the first lane was attained in the year 2002 and second lane was in 2003. In considering of this evidence, therefore the Technical Team of Tangerang - Merak Toll Road with the guidance from expert team from Japan Metropolitan Expressway decided to use the Continuously Reinforced Concrete (CRCP) overlay construction on top of the existing pavement. PT Marga Mandalasakti as the toll road investor company implemented this construction in the year 2012 until 2014 35 km length of the road use this rehabilitation method and give satisfactory results. The use of this method of construction is for the first time in Indonesia.
CONTINUOUSLY REINFORCED CONCRETE PAVEMENT (CRCP) OVERLAY CONSTRUCTION AS A SOLUTION FOR CONCRETE PAVEMENT DETERIORATION REHABILITATION IN TANGERANG-MERAK TOLL ROAD

Agung Prasetyo, Sigit Hermato

1. OVERVIEW

Tangerang - Merak Toll Road is located in the western side of Java island (Banten province), connecting the islands of Java and Sumatra. The 72.45 km length of toll road was built during the years of 1990-1996. This toll road connects Jakarta, the capital city of Indonesia to Merak, the ferry terminal port to Sumatra. The toll road has also two direction of traffic ie, the traffic direction A is the traffic toward to Merak and the direction B is to Jakarta. According to the concession agreement, PT Marga Mandalasakti is appointed as the investor of Tangerang-Merak Toll Road by government through Indonesian Toll Road Authority.

Composition of the pavement structure of Tangerang-Merak Toll Road consists of two types of pavement, the rigid pavement (Figure 2) and flexible pavement (Figure 3).

The lengths for both types of pavement structures are as shown in Table 1. And based on the implementation of the operation (the first time it was opened for traffic), the Toll Road is divided into 5 sections. Table 2 presents these details.

<table>
<thead>
<tr>
<th>DIRECTION</th>
<th>PAVEMENT LENGTH (KM)</th>
<th>TOTAL (KM)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FLEXIBLE</td>
<td>RIGID</td>
</tr>
<tr>
<td>A (Merak)</td>
<td>18.602</td>
<td>53.686</td>
</tr>
<tr>
<td>B (Jakarta)</td>
<td>17.446</td>
<td>54.288</td>
</tr>
<tr>
<td>TOTAL</td>
<td>36.048</td>
<td>107.974</td>
</tr>
</tbody>
</table>

*Source: Tangerang - Merak Toll Road Reconstruction Report (2011)*
Table 2. Toll road sections based on traffic opening

<table>
<thead>
<tr>
<th>SECTION</th>
<th>OPEN for TRAFFIC</th>
<th>LOCATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>July 1992</td>
<td>KM 26+500 – KM 47+019</td>
</tr>
<tr>
<td>II</td>
<td>March 1993</td>
<td>KM 47+019 – KM 71+929</td>
</tr>
<tr>
<td>III</td>
<td>December 1994</td>
<td>KM 71+929 – KM 87+079</td>
</tr>
<tr>
<td>IV</td>
<td>September 1995</td>
<td>KM 87+079 – KM 95+029</td>
</tr>
<tr>
<td>V</td>
<td>December 1996</td>
<td>KM 95+029 – KM 98+400</td>
</tr>
</tbody>
</table>

Source: Tangerang - Merak Toll Road Reconstruction Report (2011)

2. ROAD PAVEMENT STRUCTURE DETERIORIATION

![Figure 4. More than 20% of pavement suffered deteriorations](image)

Actually Tangerang-Merak Toll Road pavement service life time is designed for twenty years. But the structure of road pavement condition has deteriorated before the pavement life time design. This condition occurred on certain segments, especially in the constructions with rigid pavement, in several locations, the damage even has reached the worst condition.

![Figure 5. Pavement deteriorations in Tangerang - Merak Toll Road](image)
Types of the pavement deterioration that often occurs are pumping, cracks on slabs (block, edge, longitudinal, transverse). The deteriorations of flexible pavement and rigid pavement on Tangerang-Merak Toll Road are affected by two main factors:

1. The excessive of overloaded vehicle (as shown in graphic 1)
2. Retardation in handling road maintenance and rehabilitation

### THE PERCENTAGE VOLUME OF OVERLOADED VEHICLE

<table>
<thead>
<tr>
<th></th>
<th>I</th>
<th>IIA</th>
<th>IIB</th>
</tr>
</thead>
<tbody>
<tr>
<td>VEHICLE VOLUME</td>
<td>10318</td>
<td>8589</td>
<td>5362</td>
</tr>
<tr>
<td>OVERLOADED VEHICLE VOLUME</td>
<td>0</td>
<td>5644</td>
<td>3570</td>
</tr>
</tbody>
</table>

![Graphic 1: The Percentage Volume of Overload Vehicle](image)

**Figure 6. Overloaded Vehicle**

### 3. BEFORE 2009: FULL DEPTH RECONSTRUCTION METHOD

To obtain information of the cause and the proper way of handling in dealing damage in Tangerang – Merak Toll Road structures, in September 2003, *Institute of Road Engineering Agency for Research and Development of Ministry of Public Works* (PUSLITBANG) conducted weighing test surveys to measure the residual pavement service life time and obtain recommendations on handling techniques, therefore the road is still in good condition and able to deliver good services until the end of life time service design.

Based on PUSLITBANG investigation, the results were as follows: The service lifetime for the first lane of the highway toward to Jakarta has been exceeded in the year 2001, while the second lane in the year 2003. Reversely for the direction to Merak, the service lifetime for the first lane was attained in the year 2002 and second lane was in 2003.

To overcome these problems, PT. Marga Mandalasakti implemented a Partial Reconstruction of the Full Depth construction repair, by removing the damage slab and replace with the new slab then overlay with 5 cm of asphalt
pavement. Full Depth reconstructions, with modification of concrete slab reconstruction, have been done during 2002-2009. Yet the efforts, has not provided satisfactory result, the damage continuously developed from segment to another segments.

<table>
<thead>
<tr>
<th>YEAR</th>
<th>TYPE DESIGN</th>
</tr>
</thead>
<tbody>
<tr>
<td>2002</td>
<td>TYPE - 1</td>
</tr>
<tr>
<td>2003</td>
<td>TYPE - 2</td>
</tr>
<tr>
<td>2004</td>
<td>TYPE – 2, 3, 4, 5</td>
</tr>
<tr>
<td>2005</td>
<td>TYPE - 5</td>
</tr>
<tr>
<td>2006</td>
<td>TYPE – 5, 6</td>
</tr>
<tr>
<td>2007</td>
<td>TYPE – 7, 8, 9</td>
</tr>
<tr>
<td>2008</td>
<td>TYPE – 10, 11</td>
</tr>
<tr>
<td>2009</td>
<td>TYPE – 10, 12</td>
</tr>
</tbody>
</table>

Source: Tangerang Merak Toll Road Reconstruction Report (2011)

4. AFTER 2009: NEW METHOD OF RECONSTRUCTION

During 2000-2009, the road quality of Tangerang - Merak Toll Road was still poor. Marga Mandalasakti developed research in cooperated with expert team from Japan (Metropolitan Expressway). The result led to Unbounded Overlays Systems.

Unbounded Concrete Overlay is designed essentially as a new concrete pavement on stabilized sub-base, assuming an unbounded condition between the layers. Unbounded Concrete Overlay Method is thoroughly method and is considered to be more effective to be applied for more than 20% damage. This method can be applied with Jointed Reinforced Concrete Pavement (JRCP) or Continuously Reinforced Concrete Pavement (CRCP)

JRCP is jointed reinforced pavements with doweled joints at a longer spacing (typically about 9 to 12 m) and contain steel reinforcement. CRCP is a type of Portland cement concrete pavement reinforced with steel bars throughout its length.

Tangerang-Merak Toll Road reconstruction applies 2 improvement methods, which are Full Depth Reconstructions method for road structure with less than 20% damage, and Unbounded Concrete Overlay Method with JRCP / CRCP for road structure with more than 20% damage. Both methods are referred from AASHTO Design Method.

Unbounded Concrete Overlay structured with existing pavement, interlayer Asphalt Concrete (AC) Overlay, CRCP/JRCP and AC Surface Overlay (Figure 7)
5. DESIGN OF UNBOUNDED CONCRETE OVERLAY METHOD

a. AC Surface Overlay Thickness

Design parameter for AC Surface Overlay method over rigid pavement as follows (Table 5):

Table 5 Design Parameter AC Surface Overlay Over Rigid Pavement

<table>
<thead>
<tr>
<th>PARAMETER</th>
<th>Alternative 1</th>
<th>Alternative 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>- Pavement Type</td>
<td>Asphalt Concrete Resurfacing</td>
<td>asphalt Concrete Resurfacing</td>
</tr>
<tr>
<td>- Number of Lane</td>
<td>2 lanes for rigid</td>
<td>2 lanes for rigid</td>
</tr>
<tr>
<td>- Sub Grade</td>
<td>CBR = 6%</td>
<td>CBR = 6%</td>
</tr>
<tr>
<td>- Reliability Factor</td>
<td>Z = 90%</td>
<td>Z = 90%</td>
</tr>
<tr>
<td>- Concrete Characteristics</td>
<td>Concrete grade Fs = 45 kg/cm2 (equal to K-400)</td>
<td>Concrete grade Fs = 45 kg/cm2 (equal to K-400)</td>
</tr>
<tr>
<td></td>
<td>Elasticity Modulus Ec = 4,308,798 Psi</td>
<td>Elasticity Modulus Ec = 4,308,798 Psi</td>
</tr>
<tr>
<td>- Traffic Loading (West Balaraja to Ciujung)</td>
<td>37,740 .106 ESAL</td>
<td>85,957.106 ESAL</td>
</tr>
<tr>
<td>- Design Period</td>
<td>5 years</td>
<td>10 years</td>
</tr>
<tr>
<td>- Overlay Thickness</td>
<td>4cm</td>
<td>&gt;4 cm</td>
</tr>
</tbody>
</table>

Source: Tangerang Merak Toll Road Reconstruction Report (2011)

Based on above calculations, Alternative 1 is applied.
b. Unbounded JRPC/CRPC + Overlay AC Method

Design Parameter for Unbounded JRPC/CRPC + Overlay AC Method that being used are as follows:

<table>
<thead>
<tr>
<th>PARAMETER</th>
<th>VALUE</th>
</tr>
</thead>
<tbody>
<tr>
<td>- Pavement Type</td>
<td>UNBOUNDED JRPC/CRPC resurfacing</td>
</tr>
<tr>
<td>- Traffic Loading (West Balaraja to Cijung)</td>
<td>85,957.106 ESAL (10 years)</td>
</tr>
<tr>
<td></td>
<td>202,329.106 ESAL (20 years)</td>
</tr>
<tr>
<td>- Sub Grade</td>
<td>CBR = 6%</td>
</tr>
<tr>
<td>- Reliability Factor</td>
<td>Z = 90%</td>
</tr>
<tr>
<td>- Number of Lane</td>
<td>3 lanes for rigid</td>
</tr>
<tr>
<td>- Concrete Characteristics</td>
<td>Concrete grade Fs = 45 kg/cm2 (Equal to K-400)</td>
</tr>
<tr>
<td></td>
<td>Elasticity Modulus Ec = 4,308,798 Psi</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DESIGN PERIOD</th>
<th>ALTERNATIVE 1</th>
<th>ALTERNATIVE 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 years</td>
<td>23 cm</td>
<td>&gt;23 cm</td>
</tr>
<tr>
<td>20 years</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Source: Tangerang - Merak Toll Road Reconstruction Report (2011)

Based on above calculations, Alternative 1 is applied.

For cost efficiency purpose, JRPC method is chosen and built 837 m along the toll road. JRPC joints always used dowel, furthermore, cracks still form and infiltrate with water. The light steel reinforcement across these cracks is generally not enough to maintain load transfer. Then it is less common to use today.

On the other hand, CRCP is characterized by heavy steel reinforcement and absence of joint. Much more steel is used for CRCP than for JRPC. The reinforcement holds the crack tightly together and provides for aggregate interlock and shear transfer.

5 CONCLUSIONS

a. Unbounded Concrete Overlay with CRCP: an excellent rehabilitation option for concrete pavement

b. Unbounded Concrete Overlay Method with CRCP is effective for road structure with more than 20% damage

c. The performance of CRCP can be translated into cost-effective pavement solutions

d. CRCP is an effective overlay for deteriorated composite, rigid, and flexible pavement.

REFERENCES


Guide to Concrete Overlays (2008), National Concrete Pavement Technology Center, Iowa State University

Concrete Pavement Design, Construction and Performance (2008), Norbert Delatte

The IRF Vienna Manifesto on ITS: Smart Transport Policies for Sustainable Mobility

Mr. Josef Czako, Chair, IRF Policy Committee on Intelligent Transport Systems, IRF Geneva, email: jczako@irfnet.ch
Mrs. Caroline Visser, ITS Director, IRF Geneva, email: cvisser@irfnet.ch

EXECUTIVE SUMMARY
Intelligent Transport Systems (ITS) have a demonstrated ability to improve the efficiency of mobility and quality of life. ITS contributes significantly to solving today’s transport challenges such as congestion, accidents and incidents and the lack of funding for maintaining roads, in a cost-effective manner.

Policies have a major impact on the roll-out and use of intelligent technology, with many policy areas affecting deployment, such as mobility policies, environment policies and transport funding and investment policies. Political leaders can push for a wider uptake of smart technology by recognising the importance of the tools that ITS provide to address the challenges effectively and by creating the right policy framework for their deployment.

This paper explores the various policy options and provides a set of recommendations to decision makers to foster the use smart technologies to their full potential:

a) Incorporate ITS in Existing Transport Policies to acknowledge and confirm its role in achieving major transport objectives in safety, sustainability and efficiency.
b) Enhance ITS Partnerships and Collaboration between public, private and academic stakeholders to create a conducive environment for viable, user-oriented ITS services and innovation.
c) Encourage Sustainable Mobility Behaviour to balance people’s growing mobility demand with preserving the environment and quality of life.
d) Plan for ITS Deployment to create an unambiguous path towards safer, greener and more efficient roads and to embed ITS as an integral part of infrastructure planning.
e) Foster ITS Harmonisation and Standardisation to enable cross-border, user-friendly, high quality services for the public and to enable economies of scale.
f) Stimulate ITS Education to foster innovation, to enable the general public to intelligently use the transport infrastructure and the services and to continue developing the industry.

1. MAKING A CHANGE
The smart transport technologies that can allow us to make significant, positive impacts on the safety and environmental performance of our transport networks already exist in abundance. In many cases, they are already deployed and proving their worth. Unfortunately, they are not being used in anything like the scale and quantities that they need to be – especially when one considers that such technologies, collectively termed Intelligent Transport Systems or ‘ITS’, are very cost-effective and cost-conservative by comparison with traditional solutions, such as highway extensions or large scale road pricing and/or demand reduction programmes.

We face some stark choices. If left un-dealt with, we might be continuing towards:

- 1.9 million road deaths annually worldwide, by 2020, costing global society an estimated $100 billion each year.¹
- Close to 9000 Megatons of global CO2 emissions from transport vehicles by 2030², contributing significantly to climate change and, combined with other emissions, burdening millions of people with health problems through air, water, soil and noise pollution; and
- 30% increase of traffic congestion by 2025³ in some countries, costing society billions in fuel and overall economic penalties through time lost.

Or we can act now and acknowledge the role that ITS applications and services can play in addressing and managing our mobility needs and start using them to their full potential.

1.1. About this Manifesto
This Manifesto is about the role that ITS can take in improving the efficiency of our transport and, for instance, keeping the traffic on our roads moving, minimising its effects and maintaining and improving our quality of life. It is about benefits for society, both qualitative and quantitative, that can be achieved by optimising the integrated use of technology.
The planning, deployment and operation of ITS has to be people-focused and solution-driven. This Manifesto is not about technology being the right answer because it is modern. The value of ITS lies in its contribution to:

- Achieving overall transport policy objectives in terms of safety, the environment and mobility; and
- Providing access to improved mobility for all.

1.2. About the International Road Federation and its Policy Committee on ITS
The International Road Federation (IRF) brings together public, private and research sectors in the road industry. IRF promotes roads that are safe, smart and sustainable. Despite a proven track record, ITS still suffers from a lack of recognition and support from politicians, high level policy makers and the general public. IRF has taken up this awareness challenge by creating a Policy Committee on ITS, which saw its kick off in 2008.

The mission of the Committee Members is to foster the deployment of ITS. The Committee supports the development of national and regional ITS strategies and encourages governments to integrate ITS as a major tool to achieve their transport policy objectives in safety, sustainability and efficiency.

The IRF Policy Committee on ITS provides an important platform for the exchange of experience of ITS experts from all over the world on the latest progress in the development of policy frameworks and ITS action plans and strategies.

2. THE KEY: ACCESS TO MOBILITY
The ability to earn a living and so care for a family and loved ones is arguably the most basic of rights. Individuals, trade unions, other professional bodies and politicians all campaign for this on behalf of their constituencies.

There can be no economic activity without transport, which facilitates the collection of resources, the delivery of end products and all processes in between. This equates to a clear need for balanced transport networks which allow both individuals and enterprises to select the most appropriate mode, or combination of modes, to meet their needs. Access to a sustainable level of mobility can therefore be regarded as a fundamental right. It underpins all aspects of societal development allowing everyone, from individuals up to whole nations, to develop and prosper.

Indeed, lack of mobility is one of the more significant contributors to perpetuating poverty. According to the United Nations, road infrastructure density is correlated with social performance indicators. Governments, as custodians of the public good and protectors of basic rights, have an obligation to establish an enabling environment which allows all to exercise that right.

2.1 Intelligent Transport Systems (ITS) at the Service of Mobility
The options to extend infrastructure networks are often limited, due to insufficient funding, physical barriers (topography), lack of support from the general public or more complex issues around balancing multiple objectives such as the need to have liveable communities and safety. New approaches and concepts for the optimization of the transport system and for dealing with traffic demand will have to be found.

Over the last three decades a whole suite of technological solutions, together with the enabling legislative and institutional frameworks, have been developed to address the challenge of maintaining and improving mobility. ITS already has a demonstrated ability to improve mobility and quality of life by cost-effectively:

- Making traffic flow better and improving travel times;
- Making the use of transport networks more people-friendly;
- Making those networks safer; and, at the same time;
- Significantly reducing their environmental impacts.

Box 1: What are Intelligent Transport Systems?
The definition used by the ITS Policy Committee of the IRF:

Intelligent Transport Systems (ITS) apply Information and Communication Technologies (ICT) that support and optimise all modes of transport by cost-effectively improving how they work, both individually and in cooperation with each other.

This concept of ITS is made up of broad fields of application with numerous stakeholders involved:

- **Infrastructure related ITS**: applications that focus on availability and quality of transport infrastructure and that can be used to intervene in traffic capacity, to enable paying for road use, to detect incidents or hazardous weather conditions, among others.
- **Vehicle related ITS**: applications that are put in the car to support in the driver task and/or to assist in management of a fleet of vehicles.
- **User related ITS**: applications that focus on convenience and efficiency for travellers, reducing barriers to switch transport modes and provide real-time and forecast information.
- **Industry related ITS**: applications aimed at reducing costs and/or maximizing profits in the operation of transport.
- **Vehicle-to-infrastructure/vehicle-to-vehicle related ITS**: so called cooperative systems foresee the real-time interaction among vehicles and between vehicles and the road infrastructure, in order to enhance primarily traffic safety.
- **ITS back-office systems**: applications aimed at processing collected data, storing data for historic analysis, cross-application processing and system integration, providing the base for tailored, real time information flows to road managers and users.

### 2.2 Why Policy is Important for ITS Deployment

Policy is often defined as a "Statement of Intent" or a "Commitment" together with a framework for courses of action, regulatory measures, and funding priorities concerning a given topic. A policy is typically described as a set of principles or rules to guide decisions and achieve desired objectives. Policies are generally adopted by the senior governance body within an organisation or by Parliament in the case of a state, whereas procedures or protocols would be developed and adopted by senior executive officials.

The ITS Policy Committee of the IRF understands policy as:

> A set of principles and associated guidelines that provide a political, managerial, financial, and administrative framework to direct and bound actions in pursuit of explicit and prioritized goals.

Transport and mobility issues and challenges typically manifest themselves locally, but their combined impact is global. Since transport challenges are, complex and multidimensional, their solutions are cross-cutting, inter-sectoral and require a higher-order framework to find a holistic and integrating solution. Cooperation at local, regional, national and sometimes even at an international level, is required. This brings ITS into the policy arena.

To achieve the required cooperation towards solving mobility challenges, ITS deployment involves:

- Standards setting and facilitating interoperability;
- Coordination to make better use of available funding;
- Simplification of legislation and initiating new legislation if deemed necessary;
- Exchange of knowledge and sharing of best practices;
- Quantification of benefits so that the value proposition of ITS systems is better received;
- Guidance and raising awareness;

Policies have a major impact on the use of intelligent technology, as it defines the carrots and sticks guiding human behaviour. Or, as phrased by POLIS, the network of European cities and regions working together to develop innovative technologies and policies for local transport, in its 2011 paper on ITS research needs: ITS is policy-driven. The most relevant policy areas to ITS deployment are:

- Mobility policies: acknowledging the role and importance of transport and mobility for social and economic integration and development of regions, states and nations.
- Environment policies: aiming to protect the environment (air, noise, water, soil, natural habitat), to mitigate environment impacts of transport and to adapt transport infrastructure to changing conditions due to climate change.
- Transport funding and investment policies: determining the funding sources, revenues and priorities for transport infrastructure investment, maintenance and operation.
- Spatial planning policies: determining the physical grid and spacing of different functions such as living, studying, working, relaxing hence impacting on mobility needs and patterns.
- Social policies: shaping the social context in which people use transport such as accessibility, affordability, connectivity among transport modes and timing.
The following chapter relates to how ITS contributes to achieving various policy objectives. It describes the various policy challenges that governments are faced with, and highlights – based on concrete examples – how ITS can cost-effectively and efficiently contribute to achieve the policy aims.

3. HOW ITS HELPS IMPROVING…

3.1 Safe Mobility

The challenge

In its recently launched Decade of Action for Road Safety campaign, the United Nations (UN) estimates that 1.3 million people are killed on the world’s roads each year; some 20 to 50 million are injured, with many remaining disabled. The World Health Organisation (WHO) has described road casualty figures as being of ‘epidemic’ proportions. Road-related trauma is the biggest single killer of those aged between 10 and 24. The UN estimates that road accidents have an economic cost of $100 billion per year and, if unchecked, global annual road-related deaths are forecast to reach 1.9 million by 2020. The challenge is to curb this worrying trend.

The role of Intelligent Transport Systems

Road safety is commonly addressed comprehensively with the three Es of Education, Engineering and Enforcement. ITS can play a role in all of them, contributing to safer road infrastructure through clearer, more accurate warnings of danger and responses to incidents. ITS can educate by helping people to plan journeys to avoid dangerous weather conditions, by travelling at less busy times or warn of driver tiredness. Technology provides ITS based information services to road users, making them feel more at ease on the road. And increasingly innovative ways of monitoring and making sure people comply with road safety regulations, through the use of ITS, positively influences driver behaviour in traffic.

Wider deployment of ITS services and applications will result in substantial safety improvements. These technologies can increase the safety of all road users by:

- Enabling network management techniques which smooth traffic flows, reducing stop-start conditions hence reducing the variations in speed which can lead to crashes;
- Improving speed compliance and incident management such that the effects of traffic incidents, when they do occur, are minimised;
- Detecting incidents early on and shortening reaction times for appropriate rescue measures;
- In-vehicle driver support and safety applications, like for example e-Call, currently in development in the European context, where in case of an accident an automatic message is sent from the vehicle to an e-Call-centre to inform emergency services;
- Addressing the specific needs of more vulnerable road users, such as cyclists, pedestrians and those with disabilities;
- Allowing communication and interaction among vehicles and between vehicles and their surroundings which will support even greater safety; and

3.2 Transport’s Environmental Performance

The challenge

Climate change is among the major issues facing the transport sector today. According to the OECD, transport-sector CO2 emissions represent 22.5% of global man-made CO2 emissions in 2008 and account for approximately 15% of overall man-made greenhouse gas emissions. Projections state that, “under business as usual, including many planned efficiency improvements, global CO2 emissions from transport are expected to continue to grow by approximately 40% from 2007 to 2030.” The road sector dominates total transport CO2 emission production and, in a comprehensive analysis, is only surpassed by emissions from the energy production sector.

The challenge is how to effectively manage mobility while at the same time preserving the environment and quality of life.

The role of Intelligent Transport Systems

Without doubt, demand for mobility will increase and will do so at a rate which far outstrips traditional means of increasing network capacity, such as road-building. If it goes unchecked there will be significant, negative effects on carbon and other emissions related to transport.

Restricting mobility contradicts the economic benefits of free movement of people and goods and is not a sustainable solution. As alluded in Chapter 1, a certain level of mobility, or – put differently – volume of travel is essential and considered a fundamental right.
We can use ITS technologies, with their combination of information and communication technologies, to assess and potentially reduce the number of journeys which are necessary, and then reduce as far as possible the environmental footprint of those remaining. Modifying road users’ and vehicles’ behaviour in ways which make them less carbon-intensive is a much more realistic proposition than attempting to reduce demand. ITS can do this through a combination of encouragement and enforcement measures:

- **Encouragement:** ITS can be used to enable electronic payment schemes for access to certain routes, zones or facilities where the cost of access varies according to type of vehicle or time of day. This supports the ‘polluter pays’ concept, whereby those road users and vehicles with the greatest environmental impact pay the greatest share of the financial costs of putting things right. Other means of encouragement include awareness raising and sensitising (e.g. via calculation of the individual footprint) as well as measures of rewarding ecological behaviour in the form of mobility points or vouchers. Rewarding desirable mobility behaviour tends to better serve and support the general transportation policy goals than for instance granting the highest commuter tax allowances to the most frequent users of roads.

- **Enforcement:** ITS can be used for monitoring and surveillance to incur penalties and therefore mitigate excessive and aggressive driving such as speeding. It has been estimated that reductions in the speed limit with effective enforcement across the United Kingdom, would save around 1.4 Megatons in carbon emissions in the period 2009-2020. The cost of enforcement – which is envisaged would be through the monitoring of average speeds via ITS technologies – would be a factor in practical implementation.

As eco-driving relies predominantly on voluntary action – albeit substantially influenced by “carrots” and “sticks” – other measures such as fiscal incentives and education programmes may need to be introduced concurrently.

The application of ITS can benefit the environment by:

- Providing users with advanced and real-time information which allows better pre- and on-trip planning, facilitating greener choices;
- Facilitating remote financial and data transactions which improve network performance;
- Monitoring and comparing the impact on emissions from traffic management interventions;
- Realising pre-trip and en route tolling to manage demand and mitigate the effects of congestion;
- Reducing individual vehicles’ inefficient behaviour and so smooth traffic flows;
- Supporting adaptation of networks during climatic extremes by diversion and route management.

### 3.3. Traffic Flow and Travel Time Reliability

**The challenge**

The costs of traffic congestion are enormous and affect the economy, environment, public health, comfort and convenience of travellers and those living near congested networks. Figures by the OECD suggest that in 2007, Europe’s road congestion costs accumulated to 1% of its GDP, some €127 billion (equivalent to US$ 167 billion).12

The costs of urban congestion, notably in terms of travel time delays and fuel consumption, have risen sharply over the past three decades. According to the Texas Transport Institute’s 2011 Urban Mobility Report, people in the United States of America wasted an amount of 1.9 billion gallons of fuel in urban congestion, while an average commuter lost 34 hours in traffic jams in 2010. The total cost to people in 439 urban areas in the US added to US$ 115 billion.13

The 2006 Eddington Transport Study in the United Kingdom estimated that “eliminating existing congestion on the [UK’s] road network would be worth some £7.8bn (some US$ 11-12.7billion) of GDP per annum”.14 It concluded that, if left unchecked, congestion would waste an extra £22 billion (US$ 35 billion) of time in England alone by 2025.

The challenge for society is to balance supply of road infrastructure with a traffic demand that is continuing to increase and remains extremely variable.

**The role of Intelligent Transport Systems**

ITS technologies capitalise on the capabilities of computerisation, mass data storage and improved communications systems. The main functions that ITS fulfil as regards to the improvement of traffic flow and travel time reliability are:

- Enabling road operators to know what is happening on their road networks, through the collection of information about traffic speeds, volumes, incidents and accidents, to mention just a few key service
level indicators. This information plays a key role in deciding about intervening measures to smoothen and/or re-direct traffic and inform travellers.

- Reducing recurrent and non-recurrent congestion by managing traffic demand and the available road capacity. Technology provides the possibility to make road users adapt their speed to the situation on the road, to temporarilly extend the capacity of the existing road infrastructure e.g. through means the use of the hard shoulder as an extra lane, and to ration access to the motorway so as to keep traffic flowing. Many of these applications contribute to a safer driving environment as well.
- Opening the gate to a different paradigm for funding roads. Through ITS technologies, such as the electronic collection of tolls, the behaviour of road users can be incentivised, by making them pay on the base of their use of the road network. If it is more expensive to enter a certain road segment during the busiest hours, people might reconsider their travel plans.
- Providing road users with tailored real-time traffic information, before and during their trip, based on which they can adjust their travel. Journey time planners are an example of
- Enforcing traffic regulations remotely, without impact on traffic flow, an important aspect largely and increasingly supported by ITS technologies. ITS applications enable monitoring and handling compliance to, for example, speed limits and weight regulations without even having to hold or stop the vehicles involved.

One of the key developments has been the introduction of traffic or mobility centres. Mobility suggests reliable travel – and not per se the fastest travel – since its theoretical and practical application is geared towards optimising networks’ traffic flow rates. Free-flowing traffic conditions provide reliability and generally fulfill transport policy objectives by contributing to economic performance, traffic efficiency and air quality.

But can mobility centres deliver free-flowing conditions without using demand-management strategies? There may be circumstances, such as in urban areas where network capacity is severely restricted, where accompanying policies, aimed at reductions in mobility, are necessary. These strategies might, as alluded before, include the introduction of road pricing in one form or another – or, at least, stimulate reductions in the use of private transport and encouragement of the greater use of public/mass transport. A pricing strategy can set incentives to users to choose different routes, different departure times or different transport modes. A staged approach is possible and most likely in many of our cities, but there are clear benefits from individual components and systems.

Parameters for pricing strategies can be time of travel, the location, the type of vehicle used, the type of user, traffic demand or emissions, to mention just a few. Road pricing gives operators, strategists and by definition political stakeholders the flexibility to change pricing mechanisms according to the (political) needs of the moment. The more parameters used and the more data collected on them, the more refined the pricing strategy can be. The “steering” mechanism applied can be more adapted to the actual situation on the roads and enable road users, through real time information provision, to adapt their driving behavior. The ITS solutions for this are ready and available.

3.4 Innovation, Business Development and Job Creation

The challenge
The Global Financial Crisis has brought economic slowdown and recession to many countries, with many jobs lost and a decline in public purchasing power as well as in tax incomes for governments on all levels. In response, many governments have adopted severe austerity measures, cutting costs and postponing transport investment.

Adding to it the negative relation between economic growth and sustainability, the road sector is put under high pressure to come up with urgently needed innovation that cuts this irrevocable tie and that provides value-for-money investment options that improve the performance of road networks.15

The challenge for political leaders is to maximise the efficiency of existing infrastructure, provide frameworks for value-for-money technology solutions, as well as create a conducive environment for transport innovation towards safe and sustainable mobility in the near future.

The role of technology
Technology plays an important role and is a very cost-effective tool in the hands of road infrastructure managers to optimise the use of existing infrastructure. Indeed, some applications have benefit-cost ratios as high as 62 to 1.16 In times of austerity, ITS solutions can bring temporary relief to the most urgent traffic problems against relative low costs.
Additionally, investments in ITS have a so-called networking effect leading to job creation. To give an example, according to a study prepared by ITS America in 2011, the ITS end use market in the US is worth $48 billion. The report estimated that in 2009 the ITS value chain contained 445’000 private sector jobs in the US. Outlooks are bright, as job volume in the ITS value chain in the US are projected to increase to over 500’00 in 2015.\(^{17}\)

Furthermore, the use of ITS allows many of the traditional methods of managing traffic to be altogether better supported. At the same time, it allows many of these to be replaced with something more advanced. The result is a higher level of innovation overall, one which affects all of those in the transport management marketplace. Enhanced information exchange between the various market stakeholders enables:

- Greater cooperation within the market, enabling changes and improvements to logistical processes as well as the creation of innovative services and business models; and
- The creation of new products and services based on information exchange (these can include integrated transport information services, added-value and/or premium services, and advanced solutions combining several different technologies).

ITS therefore has both immediate economic advantages, in the form of the performance improvements it brings to a transport network, as well as those which may be less obvious or more longer-term in nature. For example, as data sources become more numerous and the information sets they produce become richer, so will the number of business opportunities for information service providers increase. The business models involved are very profitable and readily exportable around the world, allowing the cost of ITS deployment to be further offset.

3. 5 Implementing ITS systems

Government agencies having little experience with implementing and managing ITS systems might be daunted by the investment needed, the loss of human-human interface in traffic management due to the increase of automated intervention and the huge amount of data collected by ITS systems and that needs processing.

It helps to keep in mind that ITS systems in general have a very positive cost-benefit ratio, with some specific applications having a CB-ratio as high as 1: 60. Compared to intervening on road network level to ease congestion or improve safety, for example by expanding the through the construction of another link or lane, ITS systems are a relative affordable and effective means to achieve goals. An often-heard adagio is that one cannot build oneself out of congestion. This is particularly relevant for growing metropolitan areas where space for road traffic infrastructure is especially scarce and ITS systems can offer an effective solution for optimising the use of the existing infrastructure network.

The introduction of ITS systems does imply a changing role for road operators. With ITS systems road operators are better and quicker informed of what is happening on the road network (accidents, incidents), hence accommodating faster decision-making and relief of a situation, e.g. a quicker emergency response in case of a car accident. This can save lives. ITS systems also offer the benefit of automation, where some decisions or interventions can be triggered automatically once a set of conditions is fulfilled.

4. CONSIDERATIONS FOR THE FUTURE

4.1 The Importance of Preserving Access to Mobility

According to the United Nations, the current world population of just over 7 billion people is projected to reach 9.3 billion by 2050 and over 10 billion by 2100. Most of this increase will occur in 39 developing countries in Asia, Africa and Latin America. Urbanization prospects, also from the United Nations, suggest that the number of people living in cities and large agglomerations will increase to almost 6 out of 10 in 2030, concerning close to five billion people by that time.\(^{18}\) Rapid urbanization puts strains on urban infrastructure networks and the environment. A huge rise in the numbers of people living in cities (and mega-cities) and urban areas will lead to growing social problems – worsening traffic congestion, increasing air pollution and a growth in the numbers of road incidents.

The need for mobility will triple in 2050, compared to 2000 levels.\(^{19}\) And, as most of this increase will occur outside the developed countries, we will see a significant shift in the demand for mobility capacity. These changes pose huge challenges – but also offer great opportunities in terms of social, economic and environmental sustainability.
The shifts in transport will need new mobility concepts. Travellers will change their preferred methods of getting around, technological changes will make travel more user-friendly while at the same time making networks more resilient, and there may even be new modes of transport.

ITS will be the integration tool. It will enable local, regional and national governments in developed countries to improve already established infrastructures. It will also allow those in developing nations to leap-frog over the previous-generation networks already in place elsewhere by providing solutions which are smarter and more eco-friendly than building new road infrastructure. For densely populated urban areas it provides a tool to enhance multiple objectives, such as quality of life, public health and urban environment, preservation of historic centres next to management of road traffic and public transport.

4.2 Promising technologies

Social networking
There are already many examples of social media applications, like Facebook and Twitter, being used as the travel information source of choice during times of heavy disruption. Examples include heavy rains in Mumbai, volcanic ash clouds over Europe and more localised problems with road and rail conditions caused by snow and rain. In many such cases, traditional information sources were unable to provide travellers with the information they needed and in a timely manner.

Consumer electronics
The use of ‘probe data’ gained from tracking of both vehicles and mobile phones through the use of satellite navigation is in fact already common. Such data are made anonymous to protect citizens’ privacy and are sourced by road operators from traffic information suppliers. The data are used to supplement many types of traffic condition monitoring, both for real-time operations and historical analysis.

Such solutions can be very cost-effective for developing countries where very little investment has been made in conventional traffic monitoring equipment and for monitoring traffic movements in complex urban networks. They also underline how many of the developments in ITS technology are paralleling those in the mass-market consumer electronics sector.

Many of the traffic management, tracking, e-commerce and information applications discussed in this document can be readily supported by modern smart phones, tablets and PCs. In some cases, this means that the development of ITS solutions can be achieved at much lower costs than would have been the case only a few years ago. And, as many travellers and consumers already have their own smart portable devices, dissemination of the necessary technology and applications is already taken care of.

Connected vehicles
On-board systems in the car that are currently being developed will enable vehicles to communicate with each other and their surroundings. This interaction will bring significant benefits to our road operation.

Traffic flow can be improved. Recent tests on the A270 Motorway in the Netherlands showed that with a fleet of vehicles equipped with so-called ‘cooperative infrastructure’ technologies, a 12 to 25% improvement in traffic flow could be realised by reducing so called “shockwaves”.20

Other studies have shown that even a 10% market penetration of such technologies would start to have significant positive effects;

- Reductions in collisions, which adversely affect congestion, even if there is no injury;
- Improved responses to incidents and associated traffic management, with less delay as a result of better communications;
- Better navigation and routing, leading to significant time and fuel savings; and
- The introduction of congestion charging schemes based on precise knowledge of vehicles’ locations and prevailing traffic conditions.

Intelligent Infrastructure
The paradigm regarding transport infrastructure development and operation changes over time. In the era after World War II the main objective for governments was to reconstruct and extend their infrastructure networks, to enable economic development. The future might see a combination of economic and sustainability aspects as dominating factors in the transport infrastructure paradigm. Nowadays we might find ourselves in a transition phase between these two.21
The paradigm shift is being fed by changes and trends in travel behaviour and customer values. The Intelligent Infrastructure Studies, Foresight, UK showed for example that people start valuing predictability of travel times higher than travel time savings. Another example is the evolution from the belief that only the rich segment of society can be mobile, to mobility as a universal right, towards an attitude of “conscious” mobility.22

These changes influence the expectations from transport infrastructure networks in their capacity to accommodate mobility and their “intelligence” to adapt to changing circumstances.

Research into the 5th Generation Road (RSG), led by the French transport laboratory IFSTTAR, elaborates a system approach to demonstrate a new generation of roads, integrating promising technologies and innovation in energy, materials, information and vehicles. Four conceptual elements are being explored: the adaptable road (to lower emissions), the automated road (through the use of ICT), the resilient road (to climate change) and the acceptable road (to the public).23 Benefits are likely to kick in on service level aspects such as reliability, availability, maintainability and safety of roads.

Similar notions have been developed in the framework of the Foresight Intelligent Infrastructure Studies in the United Kingdom:
- “[A] system that can provide intelligence, with sensors and data mining providing information to support the decisions of individuals and service providers
- Infrastructure that is intelligent, processing the mass of information we collect and adapting in real time to provide the most effective services [...]”24

ITS technologies ranging from satellite based positioning to road weather information systems to real time traveller information services undoubtedly play a vital role in this trend. Taken together with the developments in vehicle-to-vehicle and vehicle-to-infrastructure interaction, we can only imagine the depth and volume of services, innovation and information that will be released and that will benefit road users to travel safe, comfortably and without delay.

5. UNLEASHING THE FULL POTENTIAL OF ITS
The previous chapters have demonstrated what value ITS can contribute in pursuing safer, greener and more efficient road travel. Promising technological trends and developments have been highlighted. The IRF Policy Committee on ITS recommends the following policy actions to support political leaders to unleash the full potential of ITS and hence societal benefits:

5.1 Incorporate ITS in Existing Transport Policies
The individual traveller cares little about an ‘intelligent transport system’; he or she cares only that the services on offer are affordable, reliable and safe. Many of the higher-level aspirations, such as reducing the environmental effects of transport, are of little concern to the end-user on a day-to-day basis.
This drives the need for policy-makers to put in place at the earliest possible opportunity a comprehensive framework, which sets out how transport might be improved. That framework, or road map, should look at the ‘how’, taking into account both political aims, such as safety, the environment and congestion reduction, and individual travellers’ needs for transport solutions that are realistic and even desirable.

From there, the necessary solutions can develop in a harmonised environment which prevents technologies and services from becoming fragmented, isolated and proprietary, and so encourages greater take-up by individuals. The end aim has to be an integrated grid of solutions that have a common ‘feel’ and recognisable levels of services.

Policy frameworks should address:
- The role of mobility in national and regional economic and social development;
- The role of technology in keeping mobility sustainable; reducing road fatalities, improving the environmental footprint of transport, reducing travel time delays due to congestion, creating jobs and innovation, and its role in funding and long term up-keeping of infrastructure;
- Accompanying policies that should come along with the introduction of smart technology, such as investments in public transport or tax incentives that encourage green mobility choices;
- Educating the general public about the intelligent use of road networks and the services available to make their journey safe and comfortable, contributing on a macro level to more sustainable travel patterns.
-
5.2 Enhance ITS Partnerships and Collaboration

As technology evolves, so do the roles of the public and private sectors and also of the research and user communities. In many cases, traditional boundaries will become blurred, especially as two-way information exchange between the public and private sectors and between road operators and road users becomes more common.

There are both challenges and opportunities.

Partnership models might be looked at to bring applications from the research to the implementation phase. There are already examples of how different parts of the transport industry have come together to trial and test applications which are now mainstream and which would not have been developed by one of the partners in isolation. It is important to note that a business case might exist on a higher level rather than that of an individual stakeholder. It implicates that societal Key Performance Indicators (KPIs) need to be matching with business KPIs, and mutual understanding of both sides of the equation should be fostered.

At a daily, operational level, the road network as a whole is shared between different authorities, often with a division between urban roads operated by city authorities and inter-urban trunk connections run by a private toll-road company or public sector highways agency. Action to address events out on the network often involves more than one jurisdiction. This division of responsibilities means that road operators must work together in concert if the full potential of connected vehicles is to be realised.

5.3 Encourage Sustainable Mobility Behaviour

Policies directed to reducing the demand of people to mobility will be less successful than policies looking to modify people’s behaviour, combined with measures to use existing infrastructure networks to their maximum potential.

ITS technologies provide the tools to translate the “user pays” principle into pragmatic pricing strategies that make people aware of their mobility patterns and the external (including environmental) costs of their travel, hence providing a grip for them to identify, and ultimately, change their mobility behaviour into a more sustainable one. The technology is there to charge people remotely and in a unified way across jurisdictions. It will also allow road transport to be compared to other transport modes.

Additionally, applying the “user pays” principle is crucial to providing new means to generate revenues that can be used to finance transport investments. In many countries, the fuel tax base is slowly but surely eroding, and with vehicles becoming more fuel-efficient, this is a trend that will not turn around. Many governments are therefore in search of sustainable funding mechanisms and ITS technology provides means to implement new revenue sources.

5.4 Plan for ITS Deployment

It is not just the actual plan itself, providing a roadmap for ITS deployment, but also the process to get to an ITS deployment strategy that can be of value. Sound analysis of the actual deployment situation in a country, state or municipality, in preparation of setting up an ITS strategy, can provide a clear image of the existing fragmentation of systems and services and can help bring about greater coherence among deployments.

Setting up a strategy should be a joint effort among public, private and academic sectors in order to achieve a common understanding of interests. It should address the institutional setting in which the plan has to be implemented, aligning the various public administrations and including an analysis of which international partners need to be informed or be involved.

Realistic timetables should be provided, by which technology and services should be delivered and deployed. The planning cycle for road investments usually typically stretches ten to 15 years. Policy makers wanting to benefit from the full potential of technology should anticipate now and embed ITS as an integral part in the planning, including long-run costs for installation, maintenance and operation. Evaluation of ITS deployment is vital to sound decision-making and should be included as a dedicated activity in the Plan.

Most importantly, a deployment plan should include a financial paragraph, stating the estimated costs and benefits and – also – who will be sharing in those costs and benefits.
5.5 Foster ITS Harmonisation and Standardisation
Harmonisation has advantages for all those involved in transportation. Strategic roads often span geographic boundaries and so common standards for data exchange and dissemination will greatly enhance cross-border (that is, trans-national) network management. Such considerations might mean little to the individual traveller, but from the network management and, ultimately, the level-of-service perspectives, there are profound implications.

For technology manufacturers, including the car-making industry, harmonisation and standardisation offers both a better understanding of what the market requires as well as economies of scale. As example: a car manufacturer can make significant savings if he can serve a global market with a single product, compared to having to tailor the product – with important modifications – to different regional markets. The same holds true for other, if not all, technology and system manufacturers.

Standardisation is equally important for the growing deployment of ITS applications in cities and metropolitan areas throughout the world. The UNECE has long-standing reputation and tradition for instance in developing standardized terminology for traffic signals; this could be transferred into new domains, such as standards and instruments associated to access restriction schemes.

Innovation and Research & Development (R&D) are essential. For some decades now the European Commission has strongly contributed to the common understanding and development of ITS across European Union Member States. In recent years, significant efforts have been made to increase levels of cooperation and harmonisation among Europe, the US and the Far East. In the future, new regions such as Latin America, Africa and other parts of Asia will achieve greater prominence and the emphasis must be on establishing cooperation from the outset.

5.6 Stimulate ITS Education
According to the UNECE there is a lack of ITS training and very limited collaboration between science, governments and industry.26 There is only a very few number of academic institutions around the world that offer integrated courses on ITS at graduate or post-graduate level. In many cases skills and competences relevant to a profession in ITS can only be obtained through combining various relevant professional domains, such as transport planning, ICT, civil engineering, public administration and policy, business administration, civil engineering, sales & marketing, to mention a few. An integrated curriculum for ITS at an academic level, preferably internationally tuned, would help in safeguarding the innovation that the ITS sector has brought forward. Collaboration between research institutes and universities, authorities and the industry is of key importance and should be stimulated.

Another important aspect is education and communication about ITS services to the general public, in order to enable intelligent use of the road networks and the information that is provided through various services.

REFERENCES

1 United Nations (2011), Improving Global Road Safety, Note by the Secretary General.
5 Statement by Mr.Ole Isaksson (Ericsson) (9 March 2011), during the 5th Working Meeting of the IRF Policy Committee on ITS, Stockholm.
6 POLIS network (2011), Research/cooperation needs for urban and regional network management and ITS, position statement, Brussels.
7 United Nations (2011), Improving Global Road Safety, Note by the Secretary-General.
8 United Nations (2011), Improving Global Road Safety, Note By UN Secretary General.
22 Blythe, Prof. Phil, Newcastle University, *Mobility scenarios for 2055: Foresight Intelligent Infrastructure Studies*, held at the Joint IRF/ITS Netherlands Workshop on Mobility 2030, Delft, 20 September 2011.
25 Statement by Mr. Dean Zabrieszach (VicRoads, Australia), during an information meeting of the IRF Policy Committee on ITS, Busan, 25 October 2010.
Construction of an Expressway Bridge Having Butterfly-Shaped Web

Kenichi Kata
Section Chief
Sumitomo Mitsui Construction Co., Ltd
Japan

Kenichi Nakatsumi
Manager
Sumitomo Mitsui Construction Co., Ltd
Japan

Naoki Maehara
Assistant Manager
West Nippon Expressway Co., Ltd., Kyushu Branch Office
Japan

Kenichiro Ashizuka
Manager
West Nippon Expressway Co., Ltd., Kyushu Branch Office
Japan

E-MAIL (for correspondence) k-kata@smcon.co.jp

KEYWORDS:
Precast panel, Fiber Reinforced Concrete, Construction Speed, Low maintenance

ABSTRACT:

It is important to reduce the superstructure weight in the earthquake prone country like Japan. Therefore, corrugated steel web bridges have been applied in many projects. However, maintenance cost should be needed to keep durability of the structure during their design life time.

A new type of bridge called “Butterfly Web Bridge” has constructed in Japan. In a butterfly web bridge, the butterfly-shaped web forms a structure that exhibits behavior similar to a double Warren truss. The 80MPa concrete is used for the butterfly web panels which are precast plate with a thickness of 150 mm. As butterfly web panels are concrete members, reinforcement provided by prestressing steels is needed on the tension side. Moreover, the 150mm plate has no reinforcement bars but is reinforced by steel fibers.

This bridge, named Takubogawa Bridge, is an expressway bridge and has 10 spans including the 87.5m maximum span length. Takubogawa Bridge is constructed by free cantilevering method. The butterfly web is prefabricated in the concrete factory and its weight is approximately 3 tons. The construction speed of cantilevering can be advanced about 50% compared with conventional concrete box girder by cast-in-situ method. This new type of bridge can meet the requirement of light weight and low maintenance.
Construction of an Expressway Bridge
Having Butterfly–Shaped Web
Kenichi Kata¹, Kenichi Nakatsumi¹, Naoki Maehara², Kenichiro Ashizuka²
¹Sumitomo Mitsui Construction Co., Ltd, Tokyo, Japan
²West Nippon Expressway Co., Ltd., Kyushu Branch Office, Fukuoka, Japan
Email for correspondence: k-kata@smcon.co.jp

1 INTRODUCTION

Takubogawa Bridge is the world’s first bridge to use a butterfly web structure ¹, ², butterfly-shaped concrete panels are utilized in the web of the main girder as a means for erecting the bridge more efficiently and for reducing construction cost. A butterfly web structure uses butterfly-shaped panels like those shown in Figure 1. With respect to shear force acting on the web, it behaves similarly to a double Warren truss structure. The material components for the web are high-strength fiber reinforced concrete and prestressing single steels used to reinforce the tensile stress areas.

![Figure 1. Behaviour of butterfly web](image1.png)

2 DESIGN OF BUTTERFLY WEB
2.1 Bridge Outline

Takubogawa Bridge (Figure 2, 3) is located in Hyuga City, Miyazaki, Japan, and forms part of the Higashi-Kyushu Expressway. The bridge perspective and general view of the structure are given below.
Structural type: 10-span continuous prestressed concrete butterfly web bridge
Length: 712.5 m Span lengths: 58.6 m - 87.5 m - 7 x 73.5 m - 49.2 m

![Figure 2. Perspective of Takubogawa Bridge](image2.png)
2.2 Materials for Butterfly Web Panels

The butterfly web consists of precast panels fabricated off-site at a plant using high strength fiber reinforced concrete with specified design strength of 80 MPa. Steel fibers of 0.2 mm diameter and 22 mm length were used to enhance the shear capacity of the panels (Figure 4). Inside the panels are prestressing steel members placed in alignment with the orientation of tension acting on the panels. Prestressing was used as the method of pretensioning. The prestressing steel components are 15.2 mm diameter strands with embossed surfaces to enhance the adhesion of concrete (Figure 5). There are no reinforcing steel, which makes the panels easy to work with and makes maintenance easy.

![Steel fibers](image1.jpg)

![Strand with embossed surface](image2.jpg)

As mentioned above, the butterfly web structure behaves similarly to a double Warren truss and shear force acts on the pinched areas of the panels, and is resisted by the shear capacity of the concrete. Consequently, the concrete used for the butterfly panels needs to have high compressive strength and shear load capacity. For this reason the design strength for compressive strength was specified as 80 MPa and the ultimate shear strength required for the pinched part of the butterfly panel was 14.3 MPa. The amount and type of fibers added was selected with the objective of raising shear strength to double that of unreinforced concrete. Table 1 shows the result of strength test between the three types of steel fibers. SW1 was made to the basis one as the mix needing the smallest volume of fibers to achieve the same shear strength.

![Table 1](table1.png)

2.3 Design of Butterfly Web Panels

Based on the main girder height and the size of the indentations that make up the butterfly shape, the butterfly web panels were designed to be 2.9 m long, and were installed at a 3.0 m pitch (Figure 6). Main girder height varies between 4.0 m and 4.5 m, but despite this variation, the panel size is kept constant over the whole bridge, reducing the portion of shear resisted by the web panels near the pier head. As described above, in terms of resistance to shear force, the behaviour of the butterfly web is similar to that of a double Warren truss. The area of tensile stress was reinforced by prestressing steel, with the amount of steel determined such that there is no tensile stress intensity under dead load, and such that no cracking occurs under design load. The structure was
designed such that the compressive force is resisted by the concrete. Web panel thickness is 150 mm, a thickness designed to be sufficient for the necessary amount of prestressing steel as described above, and to be able to resist the compressive force acting on the compression side under ultimate load. The panels incorporated dowels and steel reinforcements in order to be joined to the upper and lower deck slabs. These elements are located within a 475 mm zone at the top and bottom of each panel designed to be embedded within the concrete deck slabs. Testing has confirmed that sufficient reinforcement length should be embedded for the stressing force of the pre-tensioning steel described above to be effective in the panel.

A section of the main girder for the Takubogawa Bridge is shown in Figure 7. The butterfly web panels that comprise the web are discontinuous in the longitudinal direction of the bridge, and the panels are relatively thin. This results in the web being less rigid than that of an ordinary concrete web in a box section. Consequently, greater bending unit stress occurs in the web due to dead weight and vehicle loads. For this reason, transverse reinforcing ribs were installed at 3.0 m pitch so as to become joints between the panels, suppressing web panel deformation and reducing stress intensity.

For this bridge, in order not to produce a requirement for a large amount of prestressing steel within the butterfly web panels, diagonal external cables are installed inside the main girder in order to decrease the shear force acting on the panels. The lower deck slab of the main girder is given an inverted edge in order to be able to position these diagonal external cables at a more effective angle for reducing shear force.

As there is a discontinuous structure between the butterfly web panels, the transmission of stress intensity is complex. For this reason, testing and nonlinear analysis were performed in advance to confirm yield strength and ensure that the design would provide the prescribed shear capacity.
Using of the butterfly web means that, this bridge uses less concrete for the main girder than an ordinary concrete web box girder structure. Comparison ordinary box girder and butterfly web is shown in Table 2.

Table 2. Comparison Box girder and Bitterfly web

<table>
<thead>
<tr>
<th>Side view &amp; Section</th>
<th>Weight (superstructure)</th>
<th>Prestressing steel</th>
<th>Block length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Central Closure</td>
<td>15300kN (1.00)</td>
<td>287t (1.00)</td>
<td>3.0~4.0m</td>
</tr>
<tr>
<td>Cantilever Erection</td>
<td>130800kN (0.91)</td>
<td>233t (0.81)</td>
<td>6.0m</td>
</tr>
<tr>
<td>Column Capital</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cantilever Erection</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Central Closure</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3. CONSTRUCTION

The butterfly web panels are fabricated at a plant situated 270 km away from the bridge construction site, and transported to the site by truck. In total, the bridge requires 444 web panels, and although external shape and thickness are standardized, panels used at different points in the design require different amounts of prestressing steel and different numbers of dowels. In all, 13 different types of panel are fabricated. The fabrication process at the plant enhances efficiency by producing 4 panels of the same type at once. Since prestressing force is applied early at the fabrication stage, steam curing is used to accelerate strength gain (Figure 8, 9, 10).

Figure 8. Fabrication stage Section of the main girder

Figure 9. Steel fibers

Figure 10. Strand with embossed surface

The cantilever construction used for the Takubogawa Bridge is shown in Figure 11 and 12. Each butterfly web panel weighs approximately 3.25 t, enabling construction of the main girder lighter than would be possible with an ordinary concrete web. Consequently a construction block length of 6.0 m could be used, equivalent to the length of two butterfly web panels on each side of the bridge. As a result, whereas constructing each span of this bridge with ordinary concrete box girder sections would have required 8 blocks, the butterfly web enabled...
construction with only 5 blocks. With fewer blocks required, the construction period was substantially shortened. Also, since the butterfly web panels are not continuous in the longitudinal direction, there was no work needed to join the adjacent web elements, which also enhanced execution efficiency. On the other hand, discontinuous panels seems to have weakness point at the joint, therefore the block joints are arranged at the center of the panels, it avoids the block joint and panel gap corresponding (Figure 11).

Figure 11. Form traveller

Figure 12. Cantilever constriction

The butterfly web panels are lifted to the bridge deck by crane after transportation to the site (Figure 13), and then moved to the cantilevered deck ends where the form travelers are located. Inside the form travelers, the panels are picked up and positioned as required, and then the concrete for the upper and lower deck slabs is placed to construct the main girder. Figure 14 shows panels being put into position inside a form traveler. Central closure is shown in Figure 15. It is executed using a form traveler and then external cables are tensioned.

Figure 13. Installation of panels
Figure 14. Panels in a form traveller
4. CONCLUSIONS

In addition to enabling a lighter main girder, the butterfly web structure makes a substantial contribution to faster construction due to advantages such as requiring a smaller number of construction block increments. Piers and bearing supports can also be scaled down because of the lighter superstructure, and as a result, compared to ordinary box girder, through the reduction of the main material quantity and construction period, the cost is almost equal and the bridge has a smaller impact on the environment.

Furthermore, maintenance is easier as the web panels do not use reinforcing steel, and are of high quality products produced in a plant using industrial fabrication processes. And this structure makes it easier to inspect inside the box girder (Figure 16). Consequently, this structure provides substantial reductions in both construction costs and maintenance costs. Takubogawa Bridge construction project was completed in June 2013 and offered for use.

REFERENCES

Fiber Reinforced Asphalt Concrete: Performance Tests and Pavement Design Consideration

Kamil E. Kaloush¹, PhD, P.E., Associate Professor
Shane Underwood, PhD, Assistant Professor
Waleed Zeiada, PhD, Post-Doctoral Researcher
Jeffery Stempihar, PhD, P.E., Arizona Department of Transportation

¹Corresponding author: Department of Civil, Environmental and Sustainable Engineering, Arizona State University, Po Box 875306, Tempe, Arizona, 85287-5306, USA.
Tel (480)-965-5509; E-Mail: kaloush@asu.edu;

Abstract

Fiber Reinforced Asphalt Concrete (FRAC) mixtures continue to receive great attention from many transportation agencies world-wide because of their ability to improve pavement performance compared to conventional designs. A number of studies reported on the unique properties and characteristics of FRAC in terms of improved rutting, cracking, and raveling. Several agencies in the U.S. and countries around the world have used, or are in the process of using FRAC mixtures in new pavement designs and rehabilitation programs. This paper highlights findings from several research studies conducted at Arizona State University. The engineering properties of FRAC mixtures are discussed along with a demonstration on how to use them in current pavement design procedures.

Introduction

Modifiers in Hot Mix Asphalt (HMA) have been used to mitigate both traffic and environmentally induced pavement distresses. These include modifiers for rutting resistance by increasing the stiffness of the asphalt binders, and reduce cracking by eliminating, delaying or inhibiting crack propagation. Other benefits is to reduce or lessen the drain down of the binder for certain mixtures. There are many types of modifiers and examples include: polymers (plastomers and elastomers), fillers, and fibers. Fibers have been used to improve the performance of asphalt mixtures against pavement distresses [1, 2]. Early uses were in the 1960’s with the use of asbestos and polyester; others over the years included polypropylene, glass, carbon, coconut, cellulose and very recently aramid (or Kevlar), in addition to more than thirty recycled waste fibers that have been and continue to be introduced into the market. Early research work in Arizona, USA looked into the benefits of using tire fibers with and without crumb rubber content.

There are several research studies reporting on experiments using synthetic fibers in asphalt concrete in the literature. Bueno et al studied the addition of randomly distributed synthetic fibers on the mechanical response of a cold-mixed, densely graded asphalt mixture using the Marshall test, as well as static and cyclic triaxial tests [1]. The results showed that the addition of fibers caused small variations in the mixture’s triaxial shear strength parameters. Lee et al evaluated the influence of recycled carpet fibers on the fatigue cracking resistance of asphalt concrete using fracture energy [2]. It was found that the increase in fracture energy represents a potential for improving the asphalt mixture’s fatigue life. A research study by Fitzgerald reported that the addition of carbon fibers to an asphalt mixture may have beneficial properties ranging from improved mechanical properties to reduced electrical resistance using the electric resistivity testing methodology [3]. Cleven subjected carbon fiber-reinforced asphalt mixtures to mechanical testing, which included diametral resilient modulus, repeated load permanent deformation, flexural beam fatigue tests and indirect tensile strength tests [4]. The modified asphalt mixtures were observed to be stiffer, more resistant to permanent deformation, and had higher tensile strength at low temperatures. However, the carbon fiber modified samples showed no improvement in fatigue behavior as measured by the four point beam test or cold temperature creep compliance test. Jahromi and Khodai also investigated the characteristics and properties of the carbon fiber-
reinforced asphalt mixtures through various laboratory tests [5]. They reported that the addition of carbon fibers showed an increase in the mix’s stability, decrease in flow value, and an increase in voids in the mix. They also found that the addition of fibers improved the fatigue life and permanent deformation of the mixtures.

Mahrez and Karim utilized glass fibers in a Stone Mastic Asphalt (SMA) mixture. They found that the use of glass fiber in asphalt mixtures showed variable Marshall Stability results, and that the addition of glass fibers actually decreased the mixtures’ stability and stiffness [6]. In a different study by Mahrez and Karim in 2007, they used the wheel tracking test to characterize the creep and rutting resistance of glass fiber reinforced asphalt mixtures [7]. They reported that the inclusion of glass fibers resulted in higher resilient modulus, higher resistance to permanent strain and rutting.

Putman and Amirkhanian studied the feasibility of utilizing waste tire and carpet fibers in SMA mixtures [8]. The study compared the performance of SMA mixtures containing waste tire and carpet fibers with mixes made with commonly used cellulose and other polyester fibers. No significant difference in permanent deformation or moisture susceptibility was found in mixtures containing waste fibers compared to cellulose or polyester. However, they reported that the tire, carpet, and polyester fibers significantly improved the toughness of the mixtures compared to the cellulose fibers.

Chowdhury et al evaluated two types of recycled tire fibers to determine whether they can be used in different types of asphalt mixtures as a replacement of the currently used cellulose or mineral fibers [9]. The researchers tested three different types of mixtures: SMA, Permeable Friction Course (PFC), and Coarse Mix High Binder (CMHB) mixtures with two different types of recycled tire fibers, one cellulose fiber, and a control mix with no fibers. The laboratory tests used to evaluate the mixtures were: drain-down, dynamic modulus, indirect tensile strength, and Hamburg wheel tracking tests. Mixtures containing tire fibers, in most cases, outperformed the mixtures containing cellulose fiber and mixtures with no fiber. The drain-down test results clearly revealed that the recycled tire fiber can be used in SMA and PFC mixtures as a replacement for cellulose fibers to prevent asphalt drain-down during construction. Wu et al examined the dynamic characteristics of three fiber-modified asphalt mixtures: cellulose, polyester and mineral fibers at dosages of 0.3%, 0.3%, and 0.4% respectively [10]. The experimental results showed that fiber-modified asphalt mixtures had higher dynamic modulus compared with the control mixture.

Since 2006, Arizona State University (ASU) has been engaged in a research program to evaluate the performance benefits of synthetic fibers. The fibers are a proprietary blend of collated fibrillated polypropylene and aramid providing a three dimensional reinforcement to the HMA. The polypropylene fibers are chemically inert, non-corrosive, and non-absorbent; whereas the aramid fibers have a high tensile strength, non-corrosive and have resistance to high temperatures. The physical characteristics of the fibers have been reported in other publications [11]. The performance of the Fiber Reinforced Asphalt Concrete (FRAC) mixtures at ASU is assessed using advanced material characterization tests to assess their performance for permanent deformation and cracking. Since the polypropylene fibers play a role in binder modification, conventional consistency binder tests are also run to assess the degree of binder properties modification at different temperatures. This paper reports on some of the test results and findings of the ASU studies.

Binder Characteristics

Figure 1 shows a comparison of typical viscosity-temperature susceptibility plots for a control PG 70-10 binder and one that is modified with polypropylene (PP) fibers at the rate of 0.5 kg per tonne of asphalt mixture [12]. These plots were derived from conventional binder consistency tests including: penetration AASHTO T49-93, softening point AASHTO T53-92, and rotational viscosities at a range of temperatures AASHTO TP48. It is observed that the PP fibers improve the temperature susceptibility of the virgin binder especially at high temperatures. In fact, the small, or no-change, in viscosity at lower temperature is an advantage for the binder in keeping it on the softer side to resist thermal cracking.
Asphalt Mixture Rutting Evaluation

One test to evaluate rutting of the asphalt mixture is by conducting the Flow Number (FN) test [13]. In this test, a repeated dynamic load is applied on cylindrical specimens for several thousand repetitions, and the cumulative permanent deformation, including the beginning of the tertiary stage (FN) as a function of the number of loading cycles over the test period is recorded. Figure 2 presents FN test results conducted at 54.4 °C. The FN (inflection point in the axial strain slope) values of the FRAC mixture was found to be 10 to 15 times higher than the control mixture. It can be also observed that the control mix has higher strain slopes compared to the fiber-reinforced mixture. Lower values of strain slope during the tertiary stage means more energy is stored in the sample, and that the mix has higher potential to resist shear failure and further development of permanent deformation.

Fatigue Cracking Evaluation
Four point bending fatigue tests are common laboratory tests according to AASHTO T321-03. Each beam is subjected to a different controlled strain rate at a range of temperatures. A 50% reduction in initial stiffness is used as the criteria to determine the number of cycles until failure ($N_t$). Initial stiffness is recorded as the stiffness of the beam at 50 loading cycles in accordance with SHRP M-009 [14-16]. As an example, Figure 3 presents a fatigue comparison between the FRAC and control mixture at 4.4°C. The FRAC mixture shows performance improvement over the control mixture at 400 and 600 microstrain levels, 2 million versus 3.2 million cycles and 42,000 versus 280,000 cycles, respectively. At the 800 microstrain level, no difference in performance is observed. It is important to note that this comparison is valid since the initial stiffness of the beam samples for both mixtures analyzed was approximately 2,800 MPa. Another fatigue analysis for a different project is shown in Figure 4. It can be observed that the FRAC mixture has a higher fatigue life when compared with the control.

![Fatigue life comparison at 4.4 °C.](image1)

**Figure 3** Fatigue life comparison at 4.4 °C.

![Comparison of Control and FRAC mixtures at 21 °C.](image2)

**Figure 4** Comparison of Control and FRAC mixtures at 21 °C.

**Indirect Tensile Strength Testing**
One commonly used parameter to evaluate asphalt mixtures is tensile strength which can be used to quantify the effects of moisture and to determine the fracture resistance of an asphalt mixture. Typically, the tensile strength can be accurately determined from an indirect tensile strength test (IDT) carried out in accordance with AASHTO TP9-02 [17]. The test is conducted by applying a constant rate of vertical deformation (2.0 in/min, 50.8 mm/min) until the specimen fails. Energy until failure and total fracture energy are also calculated as the area under the stress-strain plot. Table 1 shows typical IDT test results comparing FRAC and a control mixture. It is worth mentioning that the mixture used in this project was an open graded mix. The FRAC mixture shows higher indirect tensile strength, energy at fracture and total energy than the control mixture. Depending on the test temperature, the increase in tensile strength ranges from 5 to 31%; whereas the energy at fracture range is between 15 and 100%. The contribution of the fibers is evident in the post peak strength of the material as indicated by the total energy. This improvement ranges from 19 to 45%. Although the specimen cracks, the fibers hold the specimen together which, in turn; requires more energy to completely fail the mixture. This is also demonstrated in the flexural strength test that follows.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Temperature, ( ^\circ \text{C} )</th>
<th>Indirect Tensile Strength, kPa</th>
<th>Energy at Failure, J</th>
<th>Total Energy, J</th>
</tr>
</thead>
<tbody>
<tr>
<td>FRAC</td>
<td>21.1</td>
<td>521.2</td>
<td>7.2</td>
<td>28.8</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>1077.7</td>
<td>16.8</td>
<td>53.6</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>1876.8</td>
<td>22.6</td>
<td>60.2</td>
</tr>
<tr>
<td>Laboratory Control Mix</td>
<td>21.1</td>
<td>397.1</td>
<td>5.5</td>
<td>19.8</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>951.7</td>
<td>8.3</td>
<td>37.2</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>1785.7</td>
<td>19.8</td>
<td>50.6</td>
</tr>
</tbody>
</table>

### Flexural Strength Test

As mentioned above, fibers contribute to the improvement of the load carrying capacity after the formation of the first crack in the mix. This contribution by means of bridging cracks and pull-out can be noticed by looking at the post peak region of the mechanical response of mixes (load-deformation). Under tensile stress, fibers adsorb energy preventing a dramatic propagation of cracks. Flexural strength tests are conducted to evaluate residual strength and energy characteristics of the different mixtures as shown in Figure 5. Results of flexural strength tests performed on rectangular prismatic beams of conventional and FRAC asphalt mixtures are shown below. The flexural strength of the asphalt beams is defined as the flexural stress applied on the beam at the moment of failure. The following equation is used to assess the flexural strength (FS):

\[
FS = \frac{3 \cdot F_{(peak)} \cdot L}{2 \cdot b \cdot d^2}
\]

Where,

- \( F_{(peak)} \) = peak load (lbs)
- \( L \) = length of the support span (in)
- \( b \) = width of the beam (in)
- \( d \) = thickness of the beam (in)

![Figure 5 Flexural Strength Test Set-up.](image)
As mentioned before, unlike conventional mixes, FRAC specimens do not break soon after initiation of the first crack. The fibers have the effect of increasing the work fracture which is referred to as toughness and is represented by the area under the load-deflection curve. In order to include the improvement in the material toughness imparted by the fibers the energy or work of fracture after the peak load should be included when estimating the residual strength. Banthia and Trottier presented a residual strength analysis approach on steel-fiber reinforced concrete that accounts for the toughness improvement imparted by the fibers [18]. The same approach is used to estimate the residual strength (RS) of asphalt mixes by using the following equation:

\[ RS = \frac{E_{\text{post0.25}} \cdot L}{(0.25 - \delta_{\text{peak}}) \cdot b \cdot d^2} \]

where,

- \( E_{\text{post0.25}} \) = post peak energy up to 0.25 in displacement (lb-in)
- \( \delta_{\text{peak}} \) = deflection at the peak load (in)
- \( L \) = length of the support span (in)
- \( b \) = width of the beam (in)
- \( d \) = thickness of the beam (in)

The arbitrary deflection value of 0.25 in for calculation of post peak energy is selected where every test reaches this point. Once the residual strength is estimated, it is added to the flexural strength for accounting for the improvement in toughness due to the use of fibers.

Figure 6 shows a typical load-deflection curve obtained from cyclic load test. Loading for both monotonic and cyclic load tests is controlled at a constant deflection rate of 0.025 in/min. For cyclic tests unloading is under load control at a rate of 10 lb/sec.

![Figure 6 Typical Load-Deflection Results for Cyclic Load Test](image-url)

A comparison of load-displacement curves is presented in Figure 7, where a control, FRAC mix with 1 lb/ton (~0.5 kg/tonne) and 2 lbs/ton (~1 kg/tonne) are compared. The enhancement imparted by fibers in the post peak region is noticeable. The results also suggest that a dosage of 1 lb/ton (0.5 kg/tonne) provides optimum results for the condition of this test.
C* Fracture Tests
Taking a fracture mechanics approach, a recent development of a C* Fracture Test for asphalt mixture was developed at ASU [19]. Specimens approximately 38-mm thick are cut from compacted gyratory specimens. A right-angle wedge is cut into the specimens to accommodate the loading device as shown in Figure 8. The tests can be conducted at a range of temperatures and loading rates. Once load and crack length versus time data are collected, results are analyzed and the C*-integral is plotted as a function of the crack growth rate. Even though the analysis is straightforward, a simple visual observation and assessment of the crack length versus time for two different mixture types can be ample in terms of comparing the mixtures. In Figure 8, it is observed that the FRAC mixture sample (left) will take 5.95 minutes to reach a crack length of 100 mm (4 in) compared to a control mixture (right) that will take only 3.50 minutes to reach this crack length under the specified test conditions. What is also interesting is the severity of the crack after the test. The FRAC mixture’s cracks are very tight compared to more open cracks or sample splits that are observed in control mixtures.

Dynamic Modulus Characteristics and Impact on Pavement Design
The dynamic modulus testing program follows AASHTO TP 62-07. Table 2 shows dynamic modulus values for FRAC and control mixtures at different test temperatures and frequencies. The modular ratio vary depending on test frequency and temperature, but an average modular value of 1.44 is calculated for the FRAC mixture. In the absence of any other specific dynamic modulus test data, these approximation of modular ratios (in the range of 15 to 45%) can be used as input to
determine the pavement performance using the AASHTO Mechanistic Empirical Pavement Design Guide, DARWin-ME [20]. Previous studies have shown that the performance of the FRAC mixture will be better in terms of rutting and fatigue cracking [11]. Or an alternative design strategy would be to reduce the asphalt mixture layer thickness by 30 to 40%.

Table 2 Typical Dynamic Modulus Values for Conventional and FRAC Mixtures.

<table>
<thead>
<tr>
<th>Temp. °F (°C)</th>
<th>Freq. Hz</th>
<th>Dynamic Modulus, MPa - lsi (Test Values)</th>
<th>Modular Ratio (Average 1.44)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Fiber-Reinforced</td>
<td>Conventional</td>
</tr>
<tr>
<td>14 (−10)</td>
<td>25</td>
<td>7.029</td>
<td>48.463</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>6.511</td>
<td>44.892</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>6.279</td>
<td>43.293</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>5.815</td>
<td>40.090</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>5.577</td>
<td>38.449</td>
</tr>
<tr>
<td></td>
<td>0.1</td>
<td>4.987</td>
<td>34.384</td>
</tr>
<tr>
<td>40 (4.4)</td>
<td>25</td>
<td>5.308</td>
<td>36.596</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>5.132</td>
<td>35.387</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>4.812</td>
<td>33.178</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>4.238</td>
<td>29.218</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>3.958</td>
<td>27.289</td>
</tr>
<tr>
<td></td>
<td>0.1</td>
<td>3.325</td>
<td>22.927</td>
</tr>
<tr>
<td>70 (21.1)</td>
<td>25</td>
<td>3.197</td>
<td>22.045</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>2.924</td>
<td>20.160</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>2.669</td>
<td>18.401</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>2.119</td>
<td>14.610</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>1.853</td>
<td>12.773</td>
</tr>
<tr>
<td></td>
<td>0.1</td>
<td>1.294</td>
<td>8.920</td>
</tr>
<tr>
<td>100 (37.8)</td>
<td>25</td>
<td>1.786</td>
<td>12.311</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>1.500</td>
<td>10.341</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>1.246</td>
<td>8.589</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>0.814</td>
<td>5.611</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>0.641</td>
<td>4.422</td>
</tr>
<tr>
<td></td>
<td>0.1</td>
<td>0.315</td>
<td>2.174</td>
</tr>
<tr>
<td>130 (54.4)</td>
<td>25</td>
<td>0.616</td>
<td>4.249</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>0.466</td>
<td>3.214</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>0.374</td>
<td>2.578</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>0.232</td>
<td>1.596</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>0.194</td>
<td>1.335</td>
</tr>
<tr>
<td></td>
<td>0.1</td>
<td>0.138</td>
<td>0.949</td>
</tr>
</tbody>
</table>

For AASHTO 1993 pavement design analysis with structural layer coefficient (a1) consideration, the a1 value for the fiber-reinforced mixture can be increased (by extrapolation) up to 0.53 [21]. Reminder that the layer coefficient is dependent on the resilient modulus value, which is highly correlated to the dynamic modulus property. In this approach, the analysis will also result in reduced thickness of the AC layer.

For DARWin-ME pavement design analysis, a demonstration of the process was used based on the project’s data input parameters shown in Table 3. Four main distresses were predicted in this analysis; these are: permanent deformation (rutting) of the asphalt layer, top-down fatigue cracking (longitudinal cracking), bottom-up fatigue cracking (alligator cracking), and International Roughness Index (IRI). The DARWin-ME simulations were conducted for a typical road site in Phoenix, Arizona location. The results are shown in Table 4. The results shows that the fiber reinforced mixture will reduce the potential of rutting by 40%, top-down cracking by about 72%, fatigue cracking by 42% and a resulting IRI reduction benefit of 4%. The lower IRI benefit may be something to do with the IRI models’ accuracy built in with the DARWin-ME and not necessarily a reflection of the improvement benefits of the distress.
Table 3 Input Data for DARWin- ME Analysis.

<table>
<thead>
<tr>
<th>Traffic Data</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial two-way Annual Average Daily Truck Traffic (AADTT)</td>
<td>1,500</td>
</tr>
<tr>
<td>Number of lanes in design direction</td>
<td>2</td>
</tr>
<tr>
<td>Percent of trucks in design direction (%)</td>
<td>50</td>
</tr>
<tr>
<td>Percent of trucks in design lane (%)</td>
<td>95</td>
</tr>
<tr>
<td>Operational speed, kph (mph)</td>
<td>96.6 (60)</td>
</tr>
<tr>
<td>Traffic Growth Factor</td>
<td>Comp. 4%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Climate Data</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Weather Station</td>
<td>Phoenix Airport, PHX</td>
</tr>
<tr>
<td>Latitude (degrees.minutes)</td>
<td>33.26</td>
</tr>
<tr>
<td>Longitude (degrees.minutes)</td>
<td>-111.59</td>
</tr>
<tr>
<td>Elevation, m (ft)</td>
<td>337 (1106)</td>
</tr>
<tr>
<td>Depth of water table, m (ft)</td>
<td>6.1 (20)</td>
</tr>
<tr>
<td>Mean annual air temperature, °C (°F)</td>
<td>22.92 (73.25)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Pavement Section Data</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer 1</td>
<td></td>
</tr>
<tr>
<td>Material type</td>
<td>Asphalt concrete</td>
</tr>
<tr>
<td>Layer thickness, cm (in)</td>
<td>15 (6)</td>
</tr>
<tr>
<td>Reference temperature, °C (°F)</td>
<td>21.1 (70)</td>
</tr>
<tr>
<td>Layer 2</td>
<td></td>
</tr>
<tr>
<td>Material type</td>
<td>Crushed Stone</td>
</tr>
<tr>
<td>Thickness, cm (in)</td>
<td>35 (14)</td>
</tr>
<tr>
<td>Modulus, KPa (psi)</td>
<td>172,369 (25,000)</td>
</tr>
<tr>
<td>Plasticity Index, PI</td>
<td>1</td>
</tr>
<tr>
<td>Liquid Limit, LL</td>
<td>6</td>
</tr>
<tr>
<td>Layer 3</td>
<td></td>
</tr>
<tr>
<td>Material type</td>
<td>A-6</td>
</tr>
<tr>
<td>Modulus, kPa (psi)</td>
<td>99,974 (14,500)</td>
</tr>
<tr>
<td>Plasticity Index, PI</td>
<td>16</td>
</tr>
<tr>
<td>Liquid Limit, LL</td>
<td>33</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Mixture Design Data</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt Content</td>
<td>5.5%</td>
</tr>
<tr>
<td>Air Voids</td>
<td>7.0%</td>
</tr>
<tr>
<td>VMA</td>
<td>14.6%</td>
</tr>
<tr>
<td>VFA</td>
<td>79.0%</td>
</tr>
</tbody>
</table>

Table 4 DARWin-ME Output.

<table>
<thead>
<tr>
<th>Distress Type</th>
<th>Control</th>
<th>Fiber Reinforced</th>
<th>Percent Change</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent Deformation (in)</td>
<td>0.14</td>
<td>0.1</td>
<td>40.0</td>
</tr>
<tr>
<td>AC Top-Down Cracking (Longitudinal Cracking) (ft/mile)</td>
<td>5620</td>
<td>3270</td>
<td>71.9</td>
</tr>
<tr>
<td>AC Bottom-Up Cracking (Alligator Cracking) (%)</td>
<td>17.2</td>
<td>12.1</td>
<td>42.1</td>
</tr>
<tr>
<td>IRI (in/mi)</td>
<td>126.1</td>
<td>121.0</td>
<td>4.2</td>
</tr>
</tbody>
</table>

Concluding Remarks

The blend of polypropylene and aramid fibers used in improving the engineering properties of asphalt mixtures provide substantial benefits in reducing rutting, cracking and raveling potential. The reinforcing strength contribution of fibers was evident in several mechanical tests such as the indirect tensile test, especially when the energy until fracture and total fracture energy were compared to the control mixture. The flexural strength and C* fracture tests also indicated that the FRAC mixture is better able to resist the development and propagation of cracks when compared to the control mixture. The stiffness properties measured through the dynamic modulus and used for pavement performance prediction also indicated that the FRAC will provide better rutting and fatigue cracking resistance, or as an alternative design strategy reduce the asphalt mixture layer thickness. In either context, the use of fibers make them suitable candidate as a sustainable pavement material for asphalt concrete.
References


The road network is a significant asset for Papua New Guinea and it is fair to expect that the road sector should make an important contribution to the Gross National Product. Maintaining the value of these assets places a great responsibility on the Government on PNG and maintaining the road asset is vital to any country. The PNG Road Network comprises of National Road Network of total of 8,738.47 km and over 2,000 cross drainage structures (bridges and culverts). Total road network is over 30,000 km when including National, Provincial and District Roads.

For year 2014 Department of Works (DOW) has received a very significant increase in infrastructure project funds, indeed the largest allocation for roads and bridges in the history of PNG and a total of 57 infrastructure projects are appropriated for DoW in the 2014 Budget. Of this 57 Projects, 17 were requested, 38 were not requested and 2 are from the supplementary budget.

However when stating this, the Road Asset Management System (RAMS) which is set up by Department of Works (DOW) indicates there is still much greater need for maintenance than is carried out at present. The current road maintenance budget covers approximately only 30 to 40 % of the road network needs. Spending on road maintenance and rehabilitation must be increased if the deterioration of the road network is to be halted and provide tangible benefits to the National Economy. For preparation of the maintenance plan the main assumption is to minimize total road transport costs. This is presented in the following figure. RAMS system is using this approach in preparation of periodic/specific and rehabilitation/reconstruction plans. For routine maintenance all the road sections are included not based on economics but accessibility.

To support this approach DOW has established 6 performance targets for years 2013 to 2016. These are:
Rehabilitating, Upgrading and Sealing of 1,000 km of urban, national and provincial roads at 250 km per year to recover from the maintenance backlog over the 4 years period including the Highlands Highway, Lae Nadzab, Lae City Roads, NCDC Roads, Hiritano Highway, Buluminsky Highway and Ramu Madang Highway.

Maintaining the service level of 4,000 km of national roads in good condition through well planned and coordinated routine maintenance program.

Complete ongoing projects worth K 454 million under all programs.


Revitalization of Plant and Transport Division (PTD) by implementing the commercial concept with a view to provide sustainably maintained rural access roads for local communities.

Establishing an Infrastructure Development Authority (IDA).

Performance targets for the year 2014 are based on the 2013-2016 targets and are as follows:

- 250 Km addition Km upgraded and sealed
- 3,450 km in Good Condition
- K454 million on-going projects
- West New Britain corridor opens without restriction.
- Continue to open the other 3 corridors.
- All equipment delivered and in use.

PTD procedure for coordinating the use of all Construction & Related Equipment at District level developed and being trialed in 10 Districts.

NEC policy paper on Provincial and District road maintenance program using PTD equipment.

Establish IDA

For the NTDP the following work plan has been established

<table>
<thead>
<tr>
<th>Program</th>
<th>Length (km)</th>
<th>K (millions)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GoPNG</td>
<td>26.6</td>
<td>19.1</td>
</tr>
<tr>
<td>HIP</td>
<td>67.5</td>
<td>250.8</td>
</tr>
<tr>
<td>HIP/ABG</td>
<td>72.8</td>
<td>43.8</td>
</tr>
<tr>
<td>HHWAY</td>
<td>424.1</td>
<td>122.6</td>
</tr>
<tr>
<td>NRA</td>
<td>5.0</td>
<td>6.4</td>
</tr>
<tr>
<td>NRM</td>
<td>2,666.10</td>
<td>64.0</td>
</tr>
<tr>
<td>NRM(NRA)</td>
<td>345.2</td>
<td>10.0</td>
</tr>
<tr>
<td>NRR&amp;M</td>
<td>21.8</td>
<td>9.9</td>
</tr>
<tr>
<td>Total</td>
<td>501.7</td>
<td>501.7</td>
</tr>
</tbody>
</table>

More than 85 % of the whole road network is either in a failed or poor condition category and requires major rehabilitation and reconstruction works. Concerning National road Network it has been estimated 36 per cent is in good condition. Work is underway to confirm the status of the road network through an on-going road survey program so as to underpin the priorization of road rehabilitation and reconstruction works and maintenance. The goal is to have the core road network 75 per cent to be in good condition by year 2016.
To support this DoW is focused on establishing an investment platform with a five year outlook to support the major development programs required to support growing the future of PNG by establishing a transportation network that links all of PNG. Roads are the main mode of transportation in the country and takes up more than 80 per cent of the country’s freight and passenger demand. Economic performance and effective social service delivery is unattainable without efficient mobility provided by improved road network.

The Unconstrained budget run gave budget value of 3.9 billion kina. When this is dived by provinces the largest share goes to Highlands (42 per cent). Momase and Southern have 23 per cent each and New Guinea Islands total of 12 per cent of the unconstrained run theoretical funding. This is based on the road condition needs.

As the 3.9 billion kina is not available for the budget for year 2014, the constrained budget runs has been prepared and projects and maintenance prioritized against performance targets. Based on
the constrained budget run the Projects are mainly proposed for the national priority road network (which is presented in the following figure). This is due to the reason that these roads are having majority of traffic of whole traffic volume in PNG.

Allocation for maintenance of Highlands Highway for year 2014 is 150.0 million kina. This is divided between 6 provinces. Southern Highlands receiving 53.0 million kina, Hela 7.0 million kina, Enga 14.5 million kina, Western Highlands 11.1 million kina, Jiwaka 24.8 million kina and Chimbu 23.0 million kina. Remaining 16.5 million kina has been reserved to HQ to allocate further.

Maintenance Plan including rehabilitation for National Roads is total of 100 million kina. Southern region is having total of 7.3 million kina, Highlands region 2.5 million kina, Northern region 22.4 million kina, Islands region 6.6 million kina and allocation to the HQ to be further allocated based on the needs arising in year 2014 is 25.2 million kina. This is divided for three different levels of maintenance. These are routine, periodic (specific) maintenance and rehabilitation/reconstruction maintenance.

![Allocation of Funds for Highway by Province](image)

Routine maintenance plan for National Roads for year 2014 is total of 14.2 million kina. From this allocation Southern region is receiving 1.8 million kina, Highlands region 1.1 million kina, Northern region 3.8 million kina and Islands region 2.0 million kina. 5.4 million kina has been allocated to HQ.

Periodic (Specific) maintenance plan for National Roads for year 2014 is total of 14.8 million kina. From this allocation Southern region is receiving 2.7 million kina, Highlands region 2.3 million kina, Northern region 0.9 million kina and Islands region 3.8 million kina. 5.1 million kina has been allocated to HQ.

For Rehabilitation/Reconstruction activities for National Road Network for year 2014 total budget of 71.1 million kina has been allocated. From this allocation Southern region is receiving 6.0 million kina, Highlands region 42.0 million kina, Northern region 0.9 million kina and Islands region 13.0 million kina. 5.0 million kina has been allocated to HQ.
For Bridge Maintenance Plan for the year 2014 total budget is 50 million kina. Southern region is having total of 7.7 million kina, Highlands region 11.4 million kina, Northern region 8.3 million kina, Islands region 3.5 million kina and allocation to the HQ to be further allocated based on the needs arising in year 2014 is 19.2 million kina.

Allocation for National Road Network Priority Roads is total of 64.0 million kina. Southern region is having total of 7.7 million kina, Highlands region 11.4 million kina, Northern region 8.3 million kina, Islands region 3.5 million kina and allocation to the HQ to be further allocated based on the needs arising in year 2014 is 19.2 million kina.

Following this plan, it is estimated to bring the entire public road network to a maintainable condition over a period of 15 years.
A Robust In Situ Displacement Measurement System of Bridge Structure by Using Digital Image Correlation Technique

KEYWORDS:
Displacement field measurement, bridge system, digital image correlation.

ABSTRACT:
Deflection of a bridge is a natural response of the structure due to its own weight and live load, and it has been used as the main data for the measurement of its condition. Direct measurement of bridge deflection using conventional contact methods impose practical challenge of locating the sensor and collecting the data. The challenge has motivated the development and implementation of non-contact methods for measurement of bridge deflections. Laser method, photogrammetry, and most recently DIC methods have been developed with various successes. The values of measured displacement can be used and proceed further for various purposes such as to derive strains and stresses for practical structural integrity assessment against static, fatigue or dynamic load. In the present work, a simple system to monitor and record the displacement field of the bridge system has been developed by using Digital Image Correlation (DIC) technique. The set–up to monitor the displacement field of the bridge system was developed using a digital camera, to capture the image of the bridge system when it is loaded by cars. The correlation technique was then employed off line to process the captured image data to obtain the displacement field of the bridge. Results of the full field bridge displacement obtained by using DIC technique has shown a good agreement compared with other measurement methods i.e. LVDT.
A Robust In Situ Displacement Measurement System of Bridge Structure by Using Digital Image Correlation Technique

L. Iryani\textsuperscript{2}, S. Hardono\textsuperscript{1}, H. Setiawan\textsuperscript{2}, D. Widagdo\textsuperscript{2}, T. Dirgantara\textsuperscript{2}, I.S. Putra\textsuperscript{2}

\textsuperscript{1}Institute of Road Engineering, Indonesia
\textsuperscript{2}Fakultas Teknik Mesin dan Dirgantara, Institut Teknologi Bandung, Indonesia
Email for correspondence: setyo.hardono@pusjatan.pu.go.id

ABSTRACT

Deflection of a bridge is a natural response of the structure due to its own weight and live load, and it has been used as the main data for the measurement of its condition. Direct measurement of bridge deflection using conventional contact methods imposes practical challenge of locating the sensor and collecting the data. The challenge has motivated the development and implementation of non-contact methods for measurement of bridge deflections. Laser method, photogrammetry, and most recently DIC methods have been developed with various successes. The values of measured displacement can be used and proceed further for various purposes such as to derive strains and stresses for practical structural integrity assessment against static, fatigue or dynamic load. In the present work, a simple system to monitor and record the displacement field of the bridge system has been developed by using Digital Image Correlation (DIC) technique. The set-up to monitor the displacement field of the bridge system was developed using a digital camera, to capture the image of the bridge system when it is loaded by cars. The correlation technique was then employed off line to process the captured image data to obtain the displacement field of the bridge. Results of the full field bridge displacement obtained by using DIC technique has shown a good agreement compared with other measurement methods i.e. LVDT.

INTRODUCTION

Bridges are vital infrastructure in generating and developing various social and income generating activities between places that previously unconnected or need longer distance to reach. For minimum hasssle of usage bridges are designed to have long designed life. Typical 50 to 70 years is common average lifetime of highway bridges. Although the breakdown of bridges are quite rare, the effects when it happened usually are major and even can be catastrophic in terms of casualties and financial loss. To assure the bridge safety the maintenance procedure need to be properly set up and applied. Part of the bridge maintenance is the monitoring of its deflection for the evaluation of its condition from time to time. Traditionally the deflection was measured directly using mechanical device or electrical transducers with the sensors were attached to the object at some measured points. Strain gauges or LVDT are common sensors used for this contact type of deformation measuring system. While properly set these devises can give good result, yet these kind of system give the measurement at discrete attachment points. Requirement for many points of measurement needs a lot of attached sensors accordingly. Combined with the wiring and cabling for connecting the sensors to the data acquisition and processing module, the whole system can be quite cluttered.

Recently, by the advance of digital technology, a vision based system has been implemented for bridge deflection measurement (Yoneyama et al. 2007). In the field of experimental mechanics, another vision based system, called DIC technique, has been developed to measure displacement field (Peter & Ranson 1982) (Sutton et al. 1983). The main advantage of this non-contact image based measurement is the result of the continuous full displacement field compared to discrete deformation point values obtained by the contact base system.

In this paper, study of implementing a robust on site simple monitoring and recording the displacement field of the bridge using Digital Image Correlation (DIC) technique is presented. The basic DIC measuring system that has been developed in the laboratory scale was tested for onsite realscale object measurement. The research was carried out with the support from Department of Public Works and Infrastructures Indonesia. For this initial study, they provide the access and facilities for conducting the experiment on a physical bridge model in their research and development centre. Their scope of work in planning and building the safe and supportive nationwide public infrastructures give them the interest to collaborate in this research. The set-up to monitor the displacement field of the bridge system was developed using camera of Nikon J1 to capture the images of the bridge deflection when the weight is loaded. The DIC technique was then employed off line to measure the displacement field of the 2D image captured. The accuracy was evaluated by comparing the measurement by DIC with the value from the currently accepted standard measurement tools of LVDT.

DIGITAL IMAGE CORRELATION

The first development of the DIC technique by Peters and Ranson (Peter & Ranson 1982) showed one-dimensional numerical simulations to provide initial estimation for the accuracy of deformation measurements in image
correlation. In 1983, (Sutton et al. 1983) developed 2D Digital Image Correlation (2D-DIC) method using a camera for image recording during experiment. After this pioneering works, many have developed and implemented the technique for various applications. More recently, in 2007, the 2D DIC application was done by Louis et al. (Louis et al. 2007) with the focus on measuring the deformation bands in Rothbach sandstone and Yoneyama et al. (Yoneyama et al. 2007) measure the deflection of the bridge. Dirgantara et al. (Dirgantara et al. 2010) discussed comparison of the performance of several correlation techniques and test the method for a tension specimen with an opening crack. The DIC technique was then applied in the fracture mechanics problems. (Yoneyama et al. 2007) used 2D DIC to estimate the stress intensity factor and Iryani et al. (Iryani et al. 2013) used 2D DIC to analyze the effect of DIC parameter to the stress intensity factor.

Basic of DIC techniques for measuring displacement of an object is by finding the correlation between the images of the object before and after displacement. The visual reference for the correlation process is some kind of marking at the surface of the object that can clearly be captured by camera. Using correlation function the images of marking before and after deformation is analysed to obtain the displacement value. Figure 1 illustrates the process to obtain a displacement vector at a certain point. The first step is to choose an area to be investigated, called an interrogation area in the undeformed and deformed images. Both areas must have the same size and also the same center. The next step is to choose a certain area, smaller than interrogation area, in the undeformed image at the center of the interrogation area which is defined as the subset. The investigated area in the deformed image, having the same size with the interrogation area, is defined as the template. The correlation of the subset to the template was then calculated by moving the subset pixel by pixel within the template. A matrix describing the values of correlation as a function of position in the template can be assembled as shown in Figure 1(b). The distance between one template and the adjacent template is defined as spacing. The displacement vector was determined from the center of the subset in the reference image to the location of maximum correlation result in the template. In the present work, the optimized normalized cross correlation function, which offers the most robust noise-proof performance and is insensitive to the offset and linear scale in illumination lighting as reported in (Pan et al. 2010), was used.

![Diagram of DIC process](image)

Figure 1. Schematic process to find a displacement vector, (a) correlation process (b) a matrix showing degree of similarity in the template.

**SYSTEM SET–UP AND PROCEDURE**

The equipment used in this work is a camera, the tripod to put the camera on certain position against the measured object, and a calibration sheet with checkerboard pattern. The sketch of the set–up can be seen in Figure 2. Experimental system set up with the weight and LVDT locations are shown in Figure 3. The distance of the camera to the surface of the beam is about 7.35 m.
Part of the set up procedure is the calibration process to obtain conversion value from unit pixel of the image to the unit length of the object being measured. Visual reference of marking used in the calibration process is a checkerboard pattern. The camera was placed perpendicular to the calibration sheet, and the region of interest was placed at the center of the image in order to minimize distortion. As shown in figure 4, the checkerboard used in this work has 10 mm × 10 mm square black and white pattern.
Figure 4. Checkerboard pattern

While regular pattern of checkerboard was used in the calibration, paint of a random speckle pattern was used as visual marking reference in the measurement. A lot of marking spots created by the random speckle pattern were required in the correlation process to give full field displacement value from the measurement. Figure 5(a) and Figure 5(b) show the image of area of interest before and during the loading of weight on the bridge system respectively.

(a)

(b)

Figure 5 (a) Undeformed image and (b) Deformed image of area of interest

RESULTS AND DISCUSSION

Using a 2D DIC technique, the displacement field of a large number of points in the analyzed bridge system was obtained. Figure 6(a) shows the displacement vectors of the bridge system and the contour plot of the displacement field is shown in Figure 6(b). The value of displacement measured by LVDT is used to evaluate the accuracy of displacement measurement by DIC techniques. The well comparison as shown in Table 1 indicates the good result of DIC.

(a)

(b)

Figure 6. The displacement field in observed area of the bridge system

Table 1. Displacement comparison between DIC and LVDT

<table>
<thead>
<tr>
<th>Position of Measurement</th>
<th>DIC</th>
<th>LVDT</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LVDT 1</td>
<td>6.97</td>
<td>7.20</td>
<td>3.19</td>
</tr>
<tr>
<td>LVDT 2</td>
<td>8.12</td>
<td>8.37</td>
<td>2.99</td>
</tr>
</tbody>
</table>
CONCLUSIONS

The study of implementation of image based system for onsite monitoring and recording the displacement field of the bridge system by using 2D DIC technique has been presented. The study has shown that displacement value obtained by DIC technique was in a good agreement with standard measurement using LVDT. The satisfactory difference of 3.19 % gave the confidence of this simple system of DIC can be applied for onsite measurement of the displacement of a bridge system. The displacement field obtained from the DIC technique can be processed further to obtain stress-time history which is useful for various bridge system evaluations including estimation of its remaining life.

ACKNOWLEDGEMENTS

This work was carried out in cooperation with the support from the Institute Of Road Engineering, Agency For Research and Development, Ministry of Public Works Indonesia and financial support from Program Riset Desentralisasi DIKTI 2013 & Masterplan Percepatan Pembangunan Ekonomi Indonesia (MP3EI) 2014.

REFERENCES


A Speed Restraint Pavement with a Longitudinal Surface Profile of Sine Waves

Mr Shouhei Sasaki¹, Mr Katsuaki Muraoka²

¹NIPPO CORPORATION International Division, Japan
Email for correspondence: sasaki_shouhei@nippo-c.jp

²NIPPO CORPORATION General Technology Division, Japan
Email for correspondence: muraoka_katsuaki@nippo-c.jp

1. Introduction

There are many countries that suffer increase in traffic accidents caused by drivers’ negligence, and where the number has not showed any sign of decrease so far. One of the most frequent causes is driving too fast over a speed limit, which is often seen on not only arterial roads but also community roads where old men, women and children are disposed to sacrifice. A roadway is supposed to be safe for both drivers and pedestrians, comfortable to drive, and environmentally-friendly. Therefore, these considerations must be taken into account when designing roads according to Road Structure Ordinance and specifications. But regardless of every effort by a road-agency and police, the clear improvement can’t be seen so far. Even though driving within a speed limit is a duty and an essential manner for drivers and a speed limit for streets in residential areas is generally set to secure safety of pedestrians, they are often violated without a crackdown on speeding, partly due to lack of the morality and improved performances of vehicles.

Various measures that aim to urge drivers to drop speed have been groped for, and some attempts have been tried in some countries. Adopted methods include installing “a speed bump” and “a chicane” and narrowing width. Speed bumps are devices made from various materials such as rubber, plastic, asphalt concrete, whose vertical deflection gives shocks to drivers. However, it causes not only noise but may give a damage to vehicles and endanger drivers by large shocks and jolts if passing at a high speed. In addition, a vehicle with a low ground clearance such as a sport car is hard to negotiate bumps, and it may pose serious hazards to motorcycles and bicycles if not clearly visible.

A chicane is a type of horizontal deflection by changing a road alignment and installing a short sharp turn. It also can be established not only by physically but visually using road markings. However, a chicane requires enough space to change the linearity of the road, and furthermore may cause sudden braking and large noise if drivers are not fully aware of the existence and enter the section at a high speed.

Narrowing includes reducing a road width by physical or visual methods to appeal to visual sense, making drivers feel that the road narrows and feels like slowing down by change of linearity of road markings. Physical methods involve risk of collision with oncoming vehicles and pedestrians, on the other hand, visual methods tend to take only temporary effect because a driver will become used to pass through the section after some period. In addition, three measures above are not suitable for a heavy trafficked road because they can lead to congestion and rear-end collisions. Under the circumstance described above, a safer and more effective method than conventional ones to have a driver feel like slowing down was eagerly required. The authors developed a speed restraining pavement called “Speedsave”, which eliminates the disadvantages of traditional measures. In this paper, we describe the details of Speedsave, which is an innovative way to secure safety, including a surface profile, design and construction methods, and the example of its applications and the achievements.

2. Characteristics of Speedsave

Speedsave is a new type of speed restraint pavement that features a longitudinal surface profile with continuous sine-waves. A schematic figure of Speedsave is shown in Figure 1 and 2. This unique profile is designed so as not to endanger a driver and not affect a handling for vehicles within a speed limit, and even for vehicles exceeding a speed limit. Speedsave can give a driver enough time to slow down safely without such problems caused by ordinary bumps as noise and damage. The discomfort is caused by resonance of oscillation from the suspensions, having drivers feel like slowing down, which comes from phase difference which is the angle between the vertical line to the center line and the line connecting the same points on the longitudinal curve that is shifted transversely (See Figure1).
3. Oscillation that acts on a vehicle

Oscillations caused by the surface profile are classified into three motions. (see Figure 3) Rolling is a move around the longitudinal (front-to-back) axis, while pitching is around the transversal (side-to-side) axis. Yawing is around the vertical axis. When the profile is constructed with only waves in the longitudinal direction and a vehicle passes over at a high speed, it causes not only intense pitching but also will make a vehicle jump leading to traffic accident. By transverse phase difference of a sine curve, the profile of Safepave can cause a vehicle not only longitudinal pitching motion but also rolling motion, resulting in amplifying the unpleasant feeling without endangering a driver. The combined oscillation can make a driver want to slow down even at lower speed, and make it possible to pass over at higher speed than only pitching, preventing accidents just in case that a vehicle enters carelessly at high speed.

4. Design Procedure of Speedsave

4.1. Investigation of Road Condition

The design procedure for Speedsave is shown in Figure 4. First of all, the road section where Speedsave is to be installed should be carefully investigated to get such information as road linearity, legally permitted speed, actual traveling speed, percentage of heavy vehicles and standard vehicles, the distance between intersections and so on.

4.2. Setting of Design Factors

(1) Design Speed and Maximum Speed

Setting a design speed and a maximum speed are required to design the profile of Speedsave. A design speed is a value under which drivers don’t feel unpleasant, over which drivers feel discomfort and uneasy, having them feel like slowing down. The maximum design speed is a value over which a driver feels difficulty in driving. Generally, the design speed is a regulation speed, and the maximum speed is determined considering the actual speed at which the vehicles are travelling.

For example, in the case where the regulation speed is 50km/h but the actual speed is 60 – 70 km/h, it will be expected to be proper that the design speed be set at 50km/h, the maximum speed at 80km/h to drop the average driving, the number of vehicles travelling over 80km/h.

(2) The Number of Waves

Since a profile of sine-wave for Speedsave is relatively mild, a driver may not feel unpleasant during passing over the first and the second wave. According to the test drive, it was found out that most of drivers noticed abnormality of the profile on the second wave and began to slow down after passage through the third wave. Japan’s Road Structure Ordinance provides that the time taken a driver to step on a brake is 1.5 seconds after a danger is recognized. Therefore, the minimum number of wave is obtained by the following expression, assuming a design standard that a vehicle passes through 1.5 waves per second.

\[ 3 \text{ (waves)} + 1.5 \text{ (seconds)} \times 1.5 \text{ (waves)} = 5.3 \text{ (waves)} \]

Accordingly, 6 waves are required to develop the effect of Speedsave. The smaller number of waves will not exhibit the effect because it is expected that a vehicle reaches the end point before starting to slow down. In the case that 6 to 8
waves are installed, a total length of Speedsave is about 50–60 m for the design speed of 30 km/h, 60–80 m for 40 km/h, 70–100 m for 50 km/h, 90–120 m for 60 km/h.

(3) Wave-Length
The wave-length is to be determined so that pitching of a vehicle traveling at the maximum speed is 1 to 2 Hz, given the resonance frequency above a spring of an ordinary vehicle is about 1.5 Hz.

(4) Acceleration Amplitude
Some test sections, which have each different sine-wave profile with a respective wave-length, wave-height and phase difference, were constructed in the premises of Japan Automobile Research Institute Inc. (JARI) to find a way to determine elements of the profile. The test drive was conducted at the speed of 60, 80, 100 and 120 km/h. Acceleration was measured by installing accelerometers at such 9 points as under the driver’s seat and at wheels. Pitching, a rolling angle and speed were monitored using a rate gyro installed near the center of gravity of a passenger car (sedan, 2000 cc class). In addition, sensory evaluation by drivers was conducted to assess driving feeling. The unpleasant feeling was quantified according to the five grades evaluation system shown in Table 1. Since the grade varied depending on the driver, the average grade was adopted for the final evaluation. The relationship between the acceleration and sensory evaluation was investigated to see to what extent the acceleration affects sensory evaluation. It was found out that there is high correlation between sensory evaluation and the acceleration amplitude measured under the driver’s seat with the correlation coefficient of 0.97 shown in Figure 5. As a result, it was concluded that a driver feels unpleasant at acceleration amplitude of more than 7 m/sec² (equivalent to more than grade 2) and feels quite unpleasant at more than 13 m/sec² (more than grade 3). Eventually, we set the reference acceleration amplitude for the design speed as 7 m/sec² and for the maximum speed as 12 to 15 m/sec². Figure 6 shows the example of wave-length and wave-height depending on the design speed and the maximum speed.

![Figure 5](image1)

![Figure 6](image2)

<table>
<thead>
<tr>
<th>Table 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade</td>
</tr>
<tr>
<td>0</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
</tbody>
</table>

![Design Speed and Maximum Speed](image3)
(5) Wave-Height

The wave-height is to be determined using the experimental equation developed from data at JARI so that the acceleration amplitude is within the range mentioned above for the design and maximum speed. The wave-height should be checked not to be excessively high by calculating the vertical displacements at the driver’s seat.

(6) Phase Difference

Based on the test drive, phase difference is usually set to be 20 degrees. If the difference is over the value, for example 50 degrees, large rolling is expected causing cargo shift on trucks. After determining the phase difference, rolling is to be assessed using the experimental equation in addition to empirical data. Phase difference should be adjusted and corrected depending on wave-height, since these factors mainly affect rolling.

4.3. Influence on Motorcycles

The resonance frequencies above a spring for a standard vehicle and a motorcycle are about 1.5Hz, 2.0Hz respectively. Therefore, in case that wave-length is 10m, the speed at which a vehicle oscillates the most is obtained from the following expression.

A Standard Car: 10m x 1.5Hz(1/s) x 3.6 (km/h / m/s) = 54km/h
A Motorcycle: 10m x 2.0Hz(1/s) x 3.6 (km/h / m/s) = 72km/h

The result shows that oscillation of a motorcycle above a spring is less than a standard vehicle at normal speed, and means that oscillation doesn’t lead to a fall of a motorcycle. Oscillation directly connected to a fall is called “wobble”. This motion emerges at frequency of 5-10Hz. In case that a vehicle travels at 60 km/h, the wave-length that causes this motion is obtained from the following expression.

60km/h x 3.6 (km/h / m/s) / 5 ~10Hz(1/s) = 1.7 ~ 3.3(m)

Since wave-length of speedsave is long enough compared to the calculation result above, the profile enables a motorcycle to travel safely.

5. Achievement of Speedsave

5.1. Case 1

Speedsave was constructed on a main road that passes through a large housing complex in Shinagawa Ward, Tokyo (Photo 1). This section was straight for about 800 m and was adjacent to the residential area. The percentage of a heavy vehicle was as high as 20 to 30%, and the traffic volume was as large as about 5,000 vehicles per day in each direction. The speed of traveling vehicles were measured before and 16 days after the construction of Speedsave using photoelectric speed meters. The results are shown in Table 2. Speedsave reduced the average speed by about 7 km/h and the 85 percentile speed by about 9 km/h (from 60.3 km/h before construction to 51.7 km/h after) for a standard-size car. As shown in the Figure 7, the number of vehicles traveling at 60 km/h or over was sharply reduced. The speed reduction was estimated to reduce noise by 1 to 2 dB. No change in traffic volume was observed before and after construction.

<table>
<thead>
<tr>
<th>Driving Speed (km/h)</th>
<th>Before Construction</th>
<th>After Construction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Large-Size Cars</td>
<td>Standard-Size Cars</td>
</tr>
<tr>
<td></td>
<td>Pass Number</td>
<td>Percentage</td>
</tr>
<tr>
<td>15~20</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>20~25</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>25~30</td>
<td>4</td>
<td>2</td>
</tr>
<tr>
<td>30~35</td>
<td>7</td>
<td>4</td>
</tr>
<tr>
<td>35~40</td>
<td>18</td>
<td>9</td>
</tr>
<tr>
<td>40~45</td>
<td>27</td>
<td>14</td>
</tr>
<tr>
<td>45~50</td>
<td>55</td>
<td>28</td>
</tr>
<tr>
<td>50~55</td>
<td>46</td>
<td>23</td>
</tr>
<tr>
<td>55~60</td>
<td>19</td>
<td>10</td>
</tr>
<tr>
<td>60~65</td>
<td>17</td>
<td>9</td>
</tr>
<tr>
<td>65~70</td>
<td>6</td>
<td>3</td>
</tr>
<tr>
<td>70~75</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>75~80</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>80~85</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>85~90</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>90~95</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>95~100</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>SUM</td>
<td>199</td>
<td>100</td>
</tr>
</tbody>
</table>

Average Speed

| Large-Size Cars | 49.0(km/h) | 51.8(km/h) | 43.9(km/h) | 44.6(km/h) |
| Standard-Size Cars | 51.8(km/h) | 51.8(km/h) | 45.6(km/h) | 45.6(km/h) |

948
5.2. Case 2
Reckless driving was frequent in the night with drivers enjoying running through steep curves at a high speed in the mountain road at Akagiyama in Gunma Prefecture. Speedsave (Photo 2) was constructed to eliminate such driving as intense drifting and weaving. It was reported that the average speed of 70 to 80 km/h before the construction was reduced to 50 to 60 km/h and reckless driving almost disappeared after the construction, moreover the neighborhood highly evaluated noise reduction.

5.3. Case 3
Speedsave was executed on a community road in Ayase City, Kanagawa Prefecture, aiming to reduce traffic accidents and noise (Photo 3). Vehicles had passed through the narrow road of only about 6 m in width at a high speed before the construction and caused frequent accidents near intersections. The number of traffic accidents in a year was rapidly reduced from 7 to zero after the construction, which was highly thanked by police and local communities.

6. Construction method and accuracy

6.1. Asphalt Paver
It is impossible for a conventional asphalt paver, which has a screed with one bending point at the center, to construct the profile of Speedsave with longitudinal sine waves and phase difference transversely. Therefore, the special paver of which screed is divided into 11 boards, was developed to construct the special profile (see Photo 4). Joints of each adjacent screed board can bend free to form the designed transverse profile. Elevation of each screed board is controlled by the data that is calculated based on a designed profile and put into the PC equipped with the paver. To minimize deformation during a roller compaction, double tampers-system was adopted for screed-compaction. To ensure high degree of compaction of at least 90% immediately after a placement of asphalt mixture, the paver drives at about 1 m/min.

6.2. Pneumatic Roller
When there is a phase difference transversely on the surface profile, a conventional roller deforms the designed profile formed by a paver screed during compaction. Therefore, the special pneumatic roller was developed (see Photo 5), which has 3 front and 4 rear tires that moves vertically and independently. In this way, the tires can follow the profile, not away from the surface, touching it across the width of the roller.
The comparison between the design elevation (Design Value) that is put into a PC and the measured elevation on the site (Finished Form) is shown in Figure 8. It indicates that the difference in elevation is several millimeters and negligible. Accordingly, it proved usefulness and sophistication of the developed machinery.

Figure 8

7. Precaution in applying Speedsave

There is possibility that drivers, who are driving too fast and surprised, are late in reacting when they sense large oscillation. In that case, drivers may hurriedly turn a wheel or slam on the brake, leading to causing a traffic accident. To ensure safety, some measures should be taken to let a driver perceive a waved profile before a Speedsave section begins. Among measures are road markings and a road sign for a warning. Photo 6 shows the road marking of characters saying “Pay Attention to a Waved Surface” that is installed before Speedsave zone. Photo 7 is a triangle-shaped road marking that is installed in the profile section to perceive Speedsave at a distance. Photo 8 is a road sign that shows Speedsave is ahead. But it is recommended that these measures are combined to make safety double sure.
8. Summary

Speedsave, which aims to urge drivers to drop a speed securing safety, has eliminated the disadvantages of conventional methods such as a speed bump and a chicane. Furthermore, the pavement is available for the most of roads from arterial roads to streets in a residential area. Advantages of Speedsave are as follows.
1) Within the speed limit, vehicles can travel smoothly.
2) Speed can be suppressed over a long distance.
3) It is feasible also for most of roads.
4) Noise and vibration can be reduced.
5) Bicycles and motorbikes can travel safely.
With advantages above, Speedsave has been widely implemented, and the effect has been highly appreciated by police and residents. We believe that this technology will continue to contribute to reduction in traffic accident and improvement in living environment.

REFERENCES
Shibata, 1996, “A Large-Scale Hump with a Longitudinal Surface Profile of Sine Waves”, Hosō, 31-8, Japan
PAPER TITLE
(90 Characters Max)
Countermeasure for Speed Reduction Effect in Tunnel Section by Sequence Design Study Based on Driver's Sensation

TRACK
5.3 Engineering Safer Roads

AUTHOR
(Capitalize Family Name)
Masataka Sumida
Road Sec. Group manager
IDEA Consultants, Inc.
Japan

CO-AUTHOR(S)
(Capitalize Family Name)
Sumiaki Katayanagi
Road Section manager
IDEA Consultants, Inc.
Japan
Moriaki Murata
Tunnel Section manager
IDEA Consultants, Inc.
Japan
Koichiro Yamada
Representative Director
CAPs Co., Ltd.
Japan

E-MAIL
(for correspondence)
sumida@ideacon.co.jp

KEYWORDS:
Sequence design, Traffic Driving Simulation, Traffic safety measure, Road Interior, Quantitative analysis

ABSTRACT:
It is difficult that Urban expressway in Japan has desirable road alignment because of urban land using. Shorenjugawa tunnel of Hanshin expressway Route2 - Yodogawa-Sagan route (Osaka-city, Japan) is in the same situation too. Shorenjugawa tunnel has a possibility that vehicles enter S curve exceeded speed. Accident in tunnel section is more serious than other road section. Therefore, the sequence design method that is commonly used for environmental effect in the architectural field was applied for reducing the exceeded vehicles speed at straightway before the curve. In this experimental study the sequence design is applied by patterns sequentially with gradually changing at the tunnel wall (L=350m H=2.5m) by painting. Quantitative analysis was conducted to measure the speed reduction effect and to design by driving simulation experiments and questionnaire method. And after common use of this route, actual effectiveness of this case was analyzed by using data of actual traffic. This paper will present thus designing and analyzing process in this experimental study.
A Countermeasure for Speed Reduction Effect in Tunnel Section by Sequence Design Study Based on Driver's Sensation

Masataka Sumida¹, Sumiaki Katayanagi¹, Moriaki Murata¹, Koichiro Yamada²

¹IDEA Consultants, Inc., 1-24-22 Nanko-Kita, Suminoe-Ku, Osaka, Japan
²CAPs Co., Ltd., 2-2-17 Imabashi, Chuo-Ku, Osaka, Japan

Email for correspondence: sumida@ideacon.co.jp

1 INTRODUCTION

Concerns have been raised about the risk of entering the S-shaped curve section in the Shorenjigawa Tunnel of Hanshin Expressway No. 2 Yodogawa Sagan route locate in Osaka-Japan (here in after referred to as Yodogawa Sagan route) by excessive speed. As the countermeasure of speed restraint, sequence design (here in after referred to as SQD) has been applied to the walls of the tunnel, and the best design with the highest speed restraint effect has been selected through the implementation of driving simulation experiments. In addition, this paper verified speed reduction effects about the result of the simulation and actual traffic.

2 A ROAD STRUCTURE OF YODOGAWA SAGAN ROUTE AND SPEED RESTRAINT COUNTERMEASURE

The designed speed of Yodogawa Sagan route is 60km/hr, and 80% of its structure is excavation tunnel. In particular, the eastbound of Yodogawa Sagan route, as shown in Figure 1, between the Hokko JCT and the Shorenjigawa Tunnel, is almost a straight line that heads east and transfers to underground space, where the vertical alignment changes downgrade to 1.05% from 5.0% near the Shorenjigawa Tunnel portal, even with the downgrade the road is still in a straight line, followed by the continuous S-shaped curve in Shorenjigawa Tunnel with a radius of 150m. In this way, at the long almost straight downhill, it is easy to accelerate over speed unconsciously.

Therefore in order to prevent risk, adding road markings, signs, and applying SQD in tunnel walls has been designed. This study applied SQD as enhances the effectiveness of speed restraint by using the drivers’ sense of driving. Including the multiplier effect from road markings and signs, it is possible to be an effective safety measure and to be also reducing the burden of drivers.

![Figure 1. Outline figure of the target road](image-url)
3 STUDIES ON SQD

(1) SHAPE OF PATTERN
According to the results of study on Inariyama Tunnel\(^{1,2}\), patterns of straight arrows and vertical stripes could give sense of high-speed with small sense of risk to drivers. Based on these findings, Shorenjigawa Tunnel is using straight arrows and vertical stripes as well.

(2) DISTANCE BETWEEN PATTERNS
According to the results of study on Inariyama Tunnel, it has been reported that the transit time of the most desirable pattern that balances comfort and safety is 0.272 sec / interval as shown in Figure 2. Therefore, the occurrence interval of the patterns that correspond to targeted driving speed for speed constraints has been assured.

![Figure 2. The relationship between pattern frequency and the driver’s feelings](image)

(3) LENGTH OF THE PATTERN SETTING SECTION
According to the results of the study on Inariyama Tunnel\(^{1,2}\), it has been reported that based on the average value of research result by using CG animation, drivers will get “tired” if the same pattern continues more than 8.8 seconds. In Shorenjigawa tunnel, the setting section assumes to restrain the speed from the target’s speed restrain – 70km/hr to the speed limit – 50km/hr has been arranged.

(4) METHOD OF PATTERNS DEVELOPMENT
Upon the composition of the patterns, it is necessary to restrain the speed gradually in order to avoid the risk from sudden speed change. Therefore, as shown in Figure 3, it is considered that continuously changing the shape and height of the pattern as well as occurrence interval can enhance the effect of speed restraint.

About Plan A, the total length is 465m. Used patterns are straight arrows and vertical stripes. The plan has 4 times of stage. Plan B, the total length is 342m. Used patterns are straight arrows and vertical stripes. The plan has 4 times of stage. Finally, plan C, the total length is 343m. Used pattern is only straight arrows. The plan has 3 times of stage.

![Figure 3. Schematic Drawing of Pattern Plans](image)
EVALUATION OF SQD BY THE DRIVING SIMULATION EXPERIMENT

1) EXPERIMENTAL METHOD

The driving simulation experiments (Figure 4.) was carried out to a total of 121 people including staffs from Hanshin Expressway as well as from our company by using a simple driving simulator (here in after referred to as the DS experiment). Subjects’ attribute was similar to actual drivers using Hanshin Expressway.

![Driving simulator](Image)

![Pie chart](Image)

**Fig. Driving simulator**

**Figure 4. State of DS experiment**

2) DS EXPERIMENTAL RESULTS

For the evaluation of the each proposal applied SQD by the DS experiment, comparison among unmodified original tunnel wall which painted just white colour (Plan N) and modified design (Plan A ~ C) has been conducted.

A) SPEED DECREASE AMOUNT (Table 1)

In Plan A, the speed restraint effect is observed, but it is considered that its amount is very small that in the third stage the speed restraint effect is particularly small and the design section is long due to the influence of the "tired" phenomenon. In Plan B, the intervals of patterns have gradually narrowed, as the speed restraint effect increases stage by stage. In Plan C, the decrease of speed has been receiving the impact from the design of the first stage (arrow pattern gradually increases).

<table>
<thead>
<tr>
<th></th>
<th>1st stage</th>
<th>2nd stage</th>
<th>3rd stage</th>
<th>4th stage</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Plan A</strong></td>
<td>▲0.4</td>
<td>▲0.5</td>
<td>▲0.2</td>
<td>▲0.5</td>
</tr>
<tr>
<td>total</td>
<td>▲1.6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Plan B</strong></td>
<td>▲0.6</td>
<td>▲0.6</td>
<td>▲0.9</td>
<td>▲1.2</td>
</tr>
<tr>
<td>total</td>
<td>▲3.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Plan C</strong></td>
<td>▲1.0</td>
<td>▲0.4</td>
<td>▲1.0</td>
<td></td>
</tr>
<tr>
<td>total</td>
<td></td>
<td></td>
<td></td>
<td>▲2.4</td>
</tr>
</tbody>
</table>
B) AVERAGE ACCELERATION (Figure 5)

In Plan A, from first stage to fourth stage, the average acceleration is substantially same as N draft. In the SQR section, the deceleration of Plan B and Plan C is increased uniformly than Plan N. On the other hand, the transition section is due to the deceleration is decreasing while the rapid deceleration was eased by the speed restraint in SQD setting section.

![Figure 5. AVERAGE OF ACCELERATION](image)

C) ACCELERATOR OPERATING CONDITION (Figure 6)

Plan B and C is inferred that the bottom position of accelerator is located before Plan N (at the beginning of the curve section), and Plan B and Cs’ entry into the transition section by a safe speed. In addition, Plan B and C ease the rapid deceleration, and their pedal stepping counts are looser than the Plan N. Therefore, Plan B can be called as the most effective SQD because it can prompt drivers to decelerate in transit section and ease the rapid deceleration.

![Figure 6. ACCELERATOR OPERATING CONDITION](image)
(3) EVALUATION OF OPTIMAL SQD BASED ON DS EXPERIMENTAL RESULTS

From the results above, as shown in the Table 2, Plan A, in terms of "amount of change in the average acceleration", is seen as of the decreasing speed restraint due to the "tired" = speed reduction, due to long design. By comparing the Plan B and C, in terms of "speed reduction amount", "amount of change in the average acceleration", "standard deviation of the velocity distribution", and "accelerator peak position, etc.", the effect of Plan A is observed, yet the speed restraint of Plan B is more effective. Therefore, the Plan B is adopted.

<table>
<thead>
<tr>
<th>Section</th>
<th>measurement</th>
<th>Plan A</th>
<th>Plan B</th>
<th>Plan C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Designed section</td>
<td>Speed decrease amount(km/hr) (comparison with plan N)</td>
<td>▲1.6</td>
<td>△</td>
<td>▲3.3</td>
</tr>
<tr>
<td>Curve section</td>
<td>Average of acceleration(m/s2)</td>
<td>▲0.313</td>
<td>△</td>
<td>▲0.342</td>
</tr>
<tr>
<td></td>
<td>Average of speed(km/hr)</td>
<td>74.5</td>
<td>○</td>
<td>75</td>
</tr>
<tr>
<td>Designed section</td>
<td>Peak position of accelerator (comparison with plan N)</td>
<td>60m before</td>
<td>◯</td>
<td>60m before</td>
</tr>
<tr>
<td>Designed section</td>
<td>Difference of Peak position of accelerator (comparison with plan N)</td>
<td>medium</td>
<td>○</td>
<td>large</td>
</tr>
<tr>
<td>Total Evaluation</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

5 EVALUATION OF SQD IN THE ACTUAL TRAFFIC CONDITION AFTER USAGE

Regarding to the evaluation of SQD, after analysis the actual traffic condition after usage (data of 4 hours a day of immediately after opening, about 400 cars), the unmodified design (hereinafter referred to as Plan N) and modified design (hereinafter referred to as Plan B) have been evaluated by the items of speed reduction amount and average acceleration based on the DS experiments.

(1) SPEED REDUCTION AMOUNT (Figure 7)

Compared to the speed difference in SQD section, the speed reduction amount of actual traffic is 20.7km/hr. Compared to the 17.4km/hr of Plan N(value of DS experiment) and 19.9km/hr of Plan B(value of DS experiment), the result comes out which is similar to Plan B. Looking at different stages, it is shown that in each stage of the first and second stages, the speed reduction amount of DS experiment and actual traffic condition have a similar trend which is less than 5.0km/hr.

On the other hand, when it comes to the third and subsequent stages, unlike in Plan B and actual traffic, the speed reduction ratio (inclination) of Plan N from the start of third stage to the end of fourth stage is 5%, whereas the Plan B and actual traffic has 8–10% of traffic reduction, which is almost twice of Plan N. The speed restraint effect in SQD from the third and fourth stage has been shown up.

![Figure 7. SPEED REDUCTION AMOUNT](image-url)
(2) AVERAGE ACCELERATION (Figure 8)

In the first and second stages, the actual traffic shows a slightly small deceleration than Plan B. In the third stage, although it is observed that the tendency of deceleration decreases both in Plan N and Plan B, the actual traffic maintains a deceleration equivalent to the second stage. In the fourth stage, deceleration is -0.303 (m/s²) in DS experiment and -0.493 (m/s²) in actual traffic condition. In the transition section, there is a large reduction occur in Plan N which is -0.405 (m/s²). However, the average acceleration of Plan B is -0.342 (m/s²), which is looser than Plan N. In the actual traffic, it is shown that the acceleration is reduced to the safe speed until transition section.

![Figure 8. AVERAGE ACCELERATION](image)

As a result, for the speed restraint effect in the actual traffic with installed Plan B, the effect of reducing to a speed that can enter the curve until curve section is observed. In addition, actual traffic and DS experiment shows the similar trend of acceleration and speed reduction in the tunnel. And it is suggested that the hypothetical safety measure by using driving simulation is valid.

![Figure 9. A driver’s view of the SQD tunnel](image)

6 CONCLUSIONS

By the driving simulation experiments, the sequence design is ensured the drivers’ behaviors of speed restrain as the followings:

- Throughout the design section in general, the behavior of loosening or releasing the accelerator pedal has been recognized.
- The timing of loosening the accelerator becomes earlier when entering the curve section.
- As the car entered the curve section, the peak of the pedal stepping counts is reduced.
Further, since the speed restraint effect of the sequence design selected by the driving simulation experiment shows a trend similar to the actual traffic, it is found that the hypothetical experiment by using driving simulation is effective.

For sequence design, we have already received an order for applying the concept of sequence design to diversion lane. It is expected that in the future, the sequence design will be utilized in a wide range towards the orders of proposals and safety measures.

Finally, applying SQD is varied. For example, conversely a study for speed increasing at sag point of road is expected. And SQD like this study doesn’t need electrical facilities, and is able to construct at low cost just painting. So authors expect that this study spread widely.

7 ACKNOWLEDGEMENTS

This study was based on data provided by Hanshin Expressway. We would like to express our deepest appreciation to the staff of Hanshin Expressway for providing insightful comments and suggestions.

REFERENCES


International Road Federation

Driver Behavior Education and Training Subcommittee (DBET)

Position Statement and Guidelines &
Five-Year Action Plan
(2014 – 2018)
IRF ROAD SAFETY COMMITTEE

Mission Statement

It is the mission of the Road Safety Committee (RSC) to:

1. advance the vision and mission of the International Road Federation (IRF);
2. promote the safe and efficient transportation of people and goods on urban and rural roadway systems throughout the world;
3. decrease the number of road-related crashes that result in fatalities and serious injuries;
4. reduce safety risks for motorists, bicyclists, pedestrians, and other road users; and
5. decrease the risk for property damage to businesses, industry, residential structures, and other infrastructure located adjacent to roadway systems.

Goals

The goals of the RSC are to:

1. garner road safety expertise, including best practices and newest technologies, through technology transfer from the international community to prepare and support road safety programs, policy statements, and positions;
2. educate government agencies (i.e., transportation, health, etc.), road authorities, consulting organizations, and other associations on the use of new design methods, proven technologies, and cost-effective practices;
3. influence key decision-makers to implement successful road safety strategies/programs as well as support road safety research;
4. identify focus areas where significant improvements in road safety and mobility can be achieved.

Objectives

The objectives of the DBET Subcommittee are to implement the vision, mission and goals of the RSC, specifically with the advancement of:

1. Creating a culture of “Is it Safe” when promoting driver education, training and best practice, and
2. Develop stronger links between driver behavior research, legislation and training best practice.
IRF RSC, DBET

Position Statement and Guidelines

Driver Behavior, Education and Training
# Table of Contents

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Context</td>
<td>5</td>
</tr>
<tr>
<td>Purpose</td>
<td>6</td>
</tr>
<tr>
<td>Education &amp; Training Pillars</td>
<td>7</td>
</tr>
<tr>
<td>Learner Drivers</td>
<td>9</td>
</tr>
<tr>
<td>Probationary Drivers &amp; Post License Driver Training</td>
<td>13</td>
</tr>
<tr>
<td>Early School Years &amp; Vulnerable Road Users</td>
<td>18</td>
</tr>
<tr>
<td>Adult Drivers</td>
<td>20</td>
</tr>
<tr>
<td>Endorsements</td>
<td>24</td>
</tr>
<tr>
<td>DBET Five-Year Action Plan</td>
<td>Error! Bookmark not defined.</td>
</tr>
</tbody>
</table>
Context

Until the early 1990s, driver-training “best practice” emphasized the development of psycho-motor skills and driver training practices based on procedural training. During the last 2 decades, significant research from the field of human development, psychology and cooperative efforts from road safety regulators identified the benefits of developing cognitive aspects of driving - risk perception, multiple-task performance and allocation of attention – into driver training.

In the words of Nils Gregersen (Swedish National Road and Transport Research Institute, 1997)

"safe driving is not primarily a matter of physical vehicle control and steering, but rather a matter of attitude. Increased self-awareness is a precondition of safe driving, different education strategies are needed in order to achieve these goals”.

For the effective implementation of the IRF RSC’s vision and to take a holistic “road user” approach the DBET Subcommittee shall garner and target road safety expertise specifically for driver behavior education and training for:

- Learner drivers (of eligible age and beyond), seeking professional driver training for licensing purposes
- Probationary Drivers (holding restrictive license) and fully licensed drivers seeking additional skills development (Defensive/ Advanced)
- Parents and adults “supervising” learner drivers engaged in a road safety capacity
- Management/rehabilitation/re-education of drivers who have committed traffic offences through demerit point schemes and targeted traffic offender intervention programs.

The DBET Subcommittee shall also promote road safety education (RSE) to support and create synergies with other IRF subcommittees to enhance road safety in relation to:

- School aged children to adolescents (ages 5 to 15) years, targeting traffic and road safety knowledge and awareness
- Vulnerable road users such as pedestrians, passengers, motorcyclists, cyclists.
- Target issues associated with road user behavior such as, distracted driving, drink & drug related driving, speed, fatigue, reckless driving and seat belt use.
- Law Enforcement & respecting the law as road users
- Raising awareness and understanding of new concepts between road infrastructure and road users (i.e. self-explaining roads).

Road safety research and official statistical road trauma data has well documented the hazards and risks associated for these persons and the overrepresentation of this demographic in road deaths and injuries.
Purpose

The purpose of this paper is to set policy and priorities for the DBET Subcommittee to assist the implementation the 4 key goals of the RSC, garner road safety expertise, educate government agencies, road authorities and other associations, influence key decision-makers to implement successful road safety strategies and identify focus areas where significant improvements in road safety and mobility can be achieved.
Education & Training Pillars

Context

In a learning environment people of all ages may not understand at times what is being taught. Often there are barriers to communication and learning such as individual perceptions, language, emotions, motivation, attention and preconceived notions.

Successful learning can be achieved by careful facilitation and use of knowledge and experience of participants, which should be acknowledged and applied.

People prefer to see the relevance of what they are learning in the wider context are more interested in learning to solve problems rather than gaining knowledge.

Scope

The DBET Subcommittee shall promote the following learning pillars to support and implement effective education and training that:

- Requires identifying problems and problem solving
- Defines thinking skills that shape behavior
- Determines strategies to help reduce personal risk (anticipating or avoiding danger)
- Enables “self calibration” (self-assessment and self-correction thinking)
- Promotes Hazard Perception as a key driving behavior skill
- Promotes the development of insightfulness regarding personal road safety goals and responses relating to perceptions of road safety,
- Promotes respecting road and traffic rules
- Promotes safety awareness of traffic management, road safety infrastructure and vehicle safety systems.
- Enables life skill development (information processing and decision making that is intentional)

Training & Learning Objectives

The DBET Subcommittee shall promote effective training and learning initiatives. By definition effective training and learning allows participants to:

1. acquire knowledge and principles,
2. analyze, understand and apply knowledge,
3. achieve skills (these can be self-based)
4. establish habits, and
5. develop positive attitudes

Education & Public Awareness Campaigns

The DBET Subcommittee shall promote education and awareness campaigns that promote or enable:
1. self realization and discovery (insightfulness)
2. generalization of knowledge gained and meaningful application into daily life
3. self-assessment and normalization of less risky road user behavior.
Learner Drivers

Driving Context

To assist DBET communicate and promote the above priority areas it is important that DBET defines the main competencies are required for safe driving, these are:

**Operational tasks**
Involve the direct control over the vehicle, i.e., basic vehicle maneuvers such as lane choice, lane position keeping, speed choice, braking and accelerating, starting and stopping.

**Tactical tasks**
Involve drivers interacting with other traffic whilst managing the operational task.

**Functional tasks**
Involve variations in the driving environment and pose a substantial cognitive workload. Aspects that increase cognitive workload are the interpretation and prediction of the behavior of surrounding traffic, making eye contact, deciding when to initiate a traffic maneuvers, etc.

Functional tasks involve drivers complex decision making, however this process is also influenced by personal attitudes which can shape our behaviour and reduce our cognitive performance due to emotional pressures.

Scope

The DBET Subcommittee shall promote best practice driver education and training in 3 priority areas:

1) Cognitive Skills Development
2) Attitude and Personal behavior
3) Developing competencies associated with licensing standards and solo driving

**Cognitive skills development**

Human error remains a significant contributor to road related trauma (estimated by the Federal Office of Road Safety, Australia to be as high as 95%).

Recent studies have identified significant delays in human brain development that pose challenges for training novice drivers (young drivers) and also vulnerable road users (pedestrians) and, specifically:

- Delayed development of the frontal lobes in humans takes place significantly late than other parts of the central nervous system (MRI study 2009).
- Judgment factors including the speed of moving objects, particularly in children (Wann, Poulter, & Purcell, 2011), Royal halloway University London.
Recognising and responding to hazards in living environments is a vital element for training of young drivers, pedestrians and other road users (Ivers, et al., 2006), Dr Chris Sharp, David Legge, (Monash University, Melbourne 2009)

Driver Training worldwide needs to better understand the limitations in human development particularly in the context of persons aged 18 to 28.

The development of cognitive high orders skills are a vital element in driver training and should be assessed against suitable competencies during driver training and license testing.

The graphs below whilst specific to Novice driver deaths in (Victoria, Australia 2010) and novice driver crash involvement in New South Wales, Australia, 2006-2010, are indicative of similar trends worldwide, whereby novice drivers (those with less than 5 years’ experience) are 50% more likely to be involved in a vehicle collision.

Figure 1 – TOP: Novice driver deaths in (Victoria, Australia 2010) Source Vicroads.

BOTTOM: Novice driver crash involvement in New South Wales, Australia, 2006-2010. Source NSW Centre for Road Safety, 2012
Statistically valid causation factors (gender, age, experience, habit, overconfidence and risky driving behavior) are prevalent collision factors, however the abovementioned studies identified significant new psychological and human development factors may have a greater impact. These are:

- Early adolescent (Pedestrians) under the age of 13 years cannot effectively measure and predict the speed on oncoming vehicles.

- The capacity of novice drivers to organize thought, scan and observe, prioritize risk and take appropriate corrective action is limited and not fully developed in females until the average age of 23 and males 28.

- Hazard perception training remains critical, reconfirmed earlier work by Horswill and McKenna (2004) “of all the different components of driving skill, only hazard perception has been found to relevant and consistent in identifying effective counter measures relating to crash involvement”.

The above studies established evidence that driver training for pre and post license must recognize the limitations of human development and current driver training and road safety education needs to be developed to enhance cognitive skills associated with road users.

**Attitude and Personal behavior**

**Context**

Personal attitudes shape personal behavior, and adequate knowledge is essential in forming well-balanced and informative choices.

One critical behavior for a well-balanced and informed choice is having drivers stop when law enforcement officers perform routine traffic stops. Most drivers who flee are between the ages of 13-25. Parents, educators, and the public of all ages need to teach younger drivers that police pursuits kill—that the consequences for pulling over are much less than what could happen if they flee.

The IRF DBET Subcommittee supports the position that personal characteristics are a critical element in driver training.

As a guide the DBET Subcommittee endorses the Norwegian public Roads administration -GADGET matrix (GDE) matrix which outlines essential elements of driver training including the hierarchy of driver behavior.
Licensing Standards.

Of critical importance to the integrity of road safety is the standard of licensing system applied and the level of law enforcement provided to support it. Globally many nations have adopted a safe systems approach toward road safety, with a key element being a rigorous and transparent driver licensing system.

The DBET subcommittee promotes and encourages that all nations develop, implement and enforce the following:

- Licensing & law enforcement standards that are based on best practice & competency
- Licensing curriculums and driver training is accredited
- Standards and procedures for training of persons involved with the licensing process including administration, road test examination and traffic law enforcement are transparent and accredited.
- Apply where applicable Graduated Driver licensing systems (GDLS) a system to provide new drivers of motor vehicles with driving experience and skills gradually over time in low-risk environments.

Learner Driver Competencies associated with licensing standards and solo driving

The DBET Subcommittee supports the position that less reliance is placed on short term training that helps people pass “tests” and focus on active learning methods to prepare for solo and safer driving. Specifically this involves greater training focus on:

- Establishing driving goals (safety)
- Planning and preparing for driving
- Behavioral and judgment tendencies
- Self knowledge, personality factors and personal competency
- Learning with accredited driving schools
- Learning within and respecting the licensing system and traffic laws
Probationary Drivers & Post License Driver Training

Driving Context

Whilst the challenges for safe driving are similar for learner drivers, the driving context for novice or probationary exposes them to significant increased risks as their driving context has changed from being supervised to unsupervised.

The driving context for probationary drivers can be influenced or shaped in many complex ways, however novice drivers are exposed to:

1) Increasing self-reliance to determine low risk thinking and decision making
2) Applying competencies in the real driving world,
3) Using the road within their own emotional, cognitive and behavioral development constraints, and
4) Lifestyle changes, greater independence and peer influences.

Driver Training Scope

Post licensed drivers often seeking additional driving skills (commonly referred to as defensive or Advanced driving), or additional driver training for “professional” development.

Based upon extensive supporting documentation by Elvik et al (2009), Gregersen (1996), Sanders & Keskinen (2004), and Mayhew and Simpson (2002), training programs aimed at enhancing the skills to regain control in emergency situations should not be included in basic driver education nor in post-test driver training programs.

Driver training should be aimed at improving the calibration skills of drivers. Well-calibrated drivers can detect latent hazards in traffic situations, do not underestimate the likelihood that these hazards will cause adverse effects (i.e. they are aware of the risks), and do not overestimate their own skills (i.e. they are aware of their own limitations).

The DBET Subcommittee promotes and encourages post license driver training that focuses on:

- Building resilience by improving both safety and skill level of the trainee
- Competency, not confidence
- Teaching how to avoid dangerous situations and skills to deal with unavoidable hazards
- Personality traits and how this influences risk taking
- Development of road safety goals
- Understanding the operation of vehicle safety systems (ABS, ESP, DBA, Seat belts)
- Developing cognitive skills (e.g. commentary driving)
- Apply the model (Knowledge = Transfer = Attitude = Behavior = Retention)
- Reducing distractions and maintaining concentration
- Identifying personal safety triggers.
Fundamental elements to achieve effective post license driver training

1. Supported by quality curriculum
2. Focus on the development of cognitive skills
3. Are based on “coaching” rather than teaching – involving problem solving
4. The driving context must relate to the local environment, but the training should not be context dependent (i.e. enabling students to generalise skills in the real world).

1. Quality curriculum requires:
   - A system of validation – pre and post training assessment
   - Enable knowledge transfer
   - Encourage student self-reflection
   - Is competency based
   - Enables students to demonstrate behavior change – triangulation

2. Develop Cognitive Skills (high order, thinking skills)
   - Hazard Perception - Anticipating or avoiding danger (through a combination of practice and obtaining feedback trainees can create their own understandings of how cues in traffic and a possible outcome are related. Moreover, trainees can experience the results of their own risky choices or inattention).
   - Self Calibration (self-assessment) information processing and decision making that is intentional

3. Coaching approach
   - Models include the Norwegian public Roads administration -GADGET matrix and EU Hermes coaching Project
   - Greater focus on
     - Establishing driving goals (safety)
     - Planning and preparing for driving
     - Behavioral and judgment tendencies
     - Self knowledge, personality factors and personal competency

4. Context
   - Curriculum and training must relate to the local environment, Roads, Vehicles and Collision reduction measures from statistical evidence
   - Supported by local road safety strategy
     - Police, Local Authorities, public campaigns
   - Utilise new technology to enhance competency based training
     - (simulation)
5. General:

- Courses should be varied, highly interactive, self-analytical.
- The trainer to participant ratio should be small enough to allow for individual attention and for intensive training, but large enough to facilitate stimulating group discussion.
- Practical off road exercises should be considered more of a starting point for the learning process than a complete process in itself. Each exercise should be followed with discussion.
- On road driving should be used to development cognitive skills.
- Check for undesirable side effects of training.
- Use a range of locations and teaching methods (discussions, case studies, problem-solving, self-evaluation questionnaires, videos + discussion, on-road training and driver observation, etc.)
- A good ending is vital, not rushed! and where the experiences and views of the training can be shared, summarized and discussed.
- It is not the message that is delivered, but the message which is received by the participants that counts. Constant participant feedback and course evaluation are necessary!

*See EU ADVANCED project - if a course is not extremely well designed, overestimation of one’s own skills is encouraged, and the effect on road safety will be the opposite of what is intended.

Driving Simulators & Driver Training

Studies undertaken by the University of Massechussus (2006) and the Technical University, Madrid Spain) identified that driving simulators and PC based systems are a useful learning tool when used in conjunction with scenario based training (SBT).

The Dutch Government have invested significantly in driving simulators and by late 2005, 100 simulators were used in the Netherlands in the driver training for a car driving licence (Kappé & Van Emmerik, 2005).

Dr Kappe, Mvan Emmerik, Winsum and Rozendom (Defence, security and safety, Netherlands) report that Driving simulators are a valuable tool for driver training. They allow basic vehicle handling and traffic participation to be trained effectively. Since traffic situations can be controlled, many specific training situations can be presented in a short time-span. This makes simulator training more effective than training in the real world.

Emirates Driving Company (EDC) in the United Arab Emirates, has been utilising driving simulators since 2005 as part of its internationally accredited driver training curriculum. Students take a small number of simulation lessons to target knowledge and skills that a driver needs for driving under different circumstances, aspects of driving that can increase the risk (perception of traffic situation, and speed adjustment) and critical self-evaluation of performance during training. EDC trains over 4000 students per month.

Driver training in simulators can identify functional short comings and allow students to create countermeasures to enhance cognitive thought. Examples may include:
<table>
<thead>
<tr>
<th>Element</th>
<th>Application</th>
</tr>
</thead>
</table>
| Risk perception & ordering    | Enhance HP scenarios  
Dimension-driving scenarios allow participants to identify and refine a number of cognitive skills such as:  
- Speed and lateral position measurements.  
- Approaching intersections - Recognize type of intersection in time.  
- Judge the visibility of crossing traffic - Perform scanning techniques correctly  
- Judging speed  
- Assessing complex intersections  
- Observing and predicting traffic movement, (lane changes, peripheral vision,)  
- Speed differential between slow and faster moving objects, heavy vehicles, motorcycles, pedestrians and cars)  
- Overtaking scenarios (cars, heavy vehicles)  
- Maintaining a safe following distance. |

| Self Calibration              | Expose students to a variety of traffic environments by allowing the student to control the learning environment, and to train specific driving tasks. The learning can be both simulation or on road. |

**Driver Trainer Standards**

**Assessment & qualifications**

Driver trainer standards and qualifications are important elements in maintaining adequate standards in the training system. The DBET subcommittee promotes pre assessment and selection criteria and **minimum** qualifications for driver trainers, these are:

- 3 years driving experience (excluding probationary period)
- Education level (successful completion of high school),
- Clean driving record for the previous 3 years with no major traffic offences or law violations)
- No history or conviction for drink driving or substance abuse

**Training accreditation and pedagogy**

Minimum training qualifications and training system are vital for establishing and maintaining a robust and technically competent driver training profession.

The DBET subcommittee promotes and encourages that traffic /transport authorities allow driving schools to operate under the following criteria:

- Driving schools operate under an accredited curriculum and trainee teaching method or present such documentation for endorsement to the responsible traffic / transport regulator.
- Driver trainers meet a minimum training standard (association or vocational).
The DBET subcommittee promotes and encourages driving schools to operate under a recognized training system, and this system ensures:

- Effective application of training / Learning methods
- Accreditation and training of driver trainers is based on approved professional development courses such as “train the trainer”, industry or professional driver associations, registered training organisations (RTO’s)

**Training methods**

The DBET subcommittee promotes and encourages trainers to be competent in the following training and learning principles:

1. **Practical skills:**

   - **Instructional techniques** – be able to use of a wide range of methods, lecture, group, simulation, be able to demonstrate a practical skill or example, introduce, illustrate or explain lesson material, maintain trainee interest and concentration. Learning is optimised through a combination of demonstration and participant involvement including:

     1. **Trainer demonstrates** - how to accomplish task through correct method,
     2. **Trainer links physical activity** - to knowledge and attitude
     3. **Participants repeat demonstration** - step by step,
     4. **Participants and trainer share common difficulties**,  
     5. **Instructional evaluation** – be able to measure trainee learning through test or observation, diagnose learning problems, evaluate instructor effectiveness, be able to measure the impact of learning in progress.

2. **Communication skills:**

   1. **Interpersonal / communication skills** – be able to organise information effectively, consider perceptions and feelings of students in trainee situations, use questioning in an effective manner, allow for varying needs and motivation of trainees, diagnose barriers to effective communication.
   2. **Communicating in group context** - give clear spoken instructions, use appropriate language for trainees.
Early School Years & Vulnerable Road Users

Context

No matter how well the road environment is engineered the following facts* will always remain the same:

- no-one is born knowing how to use the system
- the transport system and road environment that children inherit is complex and inherently dangerous
- the skills individuals need to manage the traffic environment are perceptual and take time to develop
- there are social, cultural, emotional and lifestyle factors over which the individual has little control or when they have choices they do so in an unsafe and anti-social way
- there will always be a need for modeling, supervision and training of children by parents and others.

* Gayle Di Pietro, GRSP RSE in schools (Malaysia 2006)

The inclusion of road safety education programs in the school curriculum is considered important and acknowledged by education systems worldwide, particularly because many crashes occur on the way to school and on the way home from school, as well as in local communities. The repercussions of children being killed or injured have a profound impact within school communities.

An effective road safety education program* involves teaching children and young people to be safer road users. It does so by developing:

- Knowledge and understanding of road traffic and the environment in which it is found
- Behavioural skills necessary to survive in the presence of road traffic
- An understanding of students’ own responsibilities for keeping themselves and others safe
- Knowledge of the causes and consequences of road accidents

According to GRSP (2000), a best practice road safety education program should:

- Begin at the pre-school level and educate continuously throughout the child’s school life
- Base the education on practical training in a realistic road environment
- Use teaching methods which follow the principles of child and adolescent development
- Base its training needs to be regular, frequent and combined with practice
- Be tailored to take account of education, cultural, transport and financial circumstances
- Have a formal place in the school curriculum
- Be reinforced by community safety schemes.

*Catchpole et al, road safety education in Schools, ARRB, Austroads.
DBET Scope

The DBET Subcommittee supports RSE in schools that promote the following.

Pre learner driver education and road safety (What approaches should schools use?).

Schools should ensure that road safety programs are:

- designed to fit within the school curriculum, are developmentally appropriate and delivered at different time points through a student’s school life rather than one-off events, talks and forums
- interactive and encourage students to develop social competence and resilience rather than purely information based programs
- part of a whole school approach including road safety policies and teacher support and training
- designed to engage with school parents and the local community given the vital role they play
- enhanced by measures to increase school connectedness among students and their parents.

What approaches should community groups use?

Community road safety groups should deliver road safety programs and campaigns that are:

- multi-action and integrated programs are delivered over time to address the complex factors
- underpinning many road safety problems
- designed to enhance and encourage a safer culture in the local community
- engaging for young people, their parents and other important community partners
- evidence-based programs, rather than approaches that intuitively feel good.

What topics should be covered?

Youth related road safety programs should inform and support:

- helping young people develop strategies to comply with the peer passenger and alcohol restrictions for novice drivers
- the importance of choosing a safe vehicle safety, especially for young drivers
- understanding and encouraging compliance with road laws
- linking with age appropriate alcohol and drug prevention programs, especially as they relate to young road user safety
- encouraging parents to be good road safety role models and providing parents with strategies to help
- reduce the risks their children face as road users
- drink driving prevention through a range of mechanisms to reduce community level access to alcohol among young people.
Adult Drivers

Scope

The DBET Subcommittee shall promote best practices to educate and change dangerous driving behaviour of adult drivers in five priority areas:

1. Observing and respecting speed limits
2. Reducing alcohol/drug use and driving
3. Increasing motorcycle helmet use
4. Increasing seat belt use
5. Reduce Distracted Driving

Context

Intentional and unintentional human error contributes to a significant percentage of traffic crashes. While there must be a vigorous approach to training new drivers, any approach to safer roads requires a comprehensive approach for all drivers. The United Nations has declared 2011-2020 a Decade of Action for Road Safety focusing on certain behavioural risk factors that could significantly reduce traffic crashes and/or fatalities. The above list contains a number of those risky activities, with the issue of Distracted Driving as an additional new concern. To effectively change adult behaviour and curtail these risky actions, three important factors must be present:

- strong laws that are transparent in their benefit
- effective messaging of the laws and the purpose behind them to effectively engage with road users
- rigorous enforcement of laws.

Without the laws, the behaviour is not improper; without the messaging there is no understanding of the reason for the laws; without the enforcement there are no consequences thus no reason to change, education alone is usually insufficient.¹ It takes all three factors combined to change behaviour.

Effective messaging involves an effective road safety education program in schools (i.e., prior to driving), public education campaigns that integrate with police enforcement activities, and effective policies to manage/rehabilitate/re-educate those drivers who have commit traffic offences (through demerit point schemes and targeted traffic offender intervention programs).

Overall, using this three-step process changes behaviour of most drivers and creates a safer road user culture. The DBET Subcommittee endorses the U.N.’s risk factors as critical in creating safer roads for all.

Increasing Road Safety Legislation

While many countries have separately passed legislation dealing with seat belts, substance abuse and driving, or motorcycle helmets, it is the comprehensive approach on all of these topics that is

¹ Improving the Effectiveness of Road Safety Campaigns: Current and New practices, Hoekstra, T., Wegman, F. 2010.
missing. As of 2013, ninety-four countries have laws that address these factors in some fashion, but only twenty-eight countries have comprehensive laws on all five U.N. risky behaviours.2

To aid governments’ understanding of what laws are needed, and based on an examination of the successful countries the DBET Subcommittee will develop (1) a comprehensive checklist and promote it to:

• government agencies,
• legislators,
• safety advocates, and
• community leaders

This will allow for others to determine the missing elements from their own country’s jurisprudence.

Additionally, the DBET Subcommittee will (2) supply with the checklist research supporting these actions, such as statistics demonstrating the benefit of wearing seat belts. This can provide an scientific basis for the implementation of these laws.

In an effort to support active participation of all segments in a community, the DBET Subcommittee will promote activities that educate government agencies, legislators, safety advocates, and community leaders on what is needed for an effective statutory structure in traffic safety. The focus will be on the five priority areas listed above: reducing speed, reducing alcohol/drug use and driving, increasing motorcycle helmet use, increasing seat belt use, and distracted driving.

**Effective Messaging**

A road safety message that works in one country may not work at all in another country. With different cultures, different issues, and a variety of communication methods it is not likely a one-size fits all message can or should be developed. Local entities need to be allowed to “buy” into the message, thus developing their own messages will be more effective. What can be created are tools describing effective communication, e.g. basic principles that can be examined and shared.

Tools currently exist to inform and support traffic safety messaging, such as *Road Safety Communication Campaigns, Manual for Design, Implementation and Evaluation*, Delhomme, P. et. al. 20103; *White Paper on Traffic Safety Culture*, Ward, N. et. al. (2010) and a variety of others.

Using the information in existence, the DBET Subcommittee can assess, and develop a variety of targeted methods currently used and distribute basic principles from those methods.

**Rigorous Enforcement**

When laws are not enforced, behaviour is not changed. Unfortunately, many law enforcement agencies (and officers) look at traffic safety as a tangential requirement of “protect and serve”,

---

2 The five topics include: speed, reducing drinking and driving, increasing motorcycle helmet use, increasing seat belt use, and increasing the use of child restraints.
3 More information can be found this publication and about CAST—*Campaigns and Awareness-raising Strategies in Traffic Safety* at: http://www.cast-eu.org/pages/publications.html
taking a “back seat” to other law enforcement activities. Yet when looking at top ten leading global
causes of death, excluding chronic diseases, traffic crashes is a primary cause of death. Similar to
many chronic diseases, traffic crashes are preventable with targeted interventions.\(^4\)

While governments can typically do more to support law enforcement agencies (e.g. budgetary
support, hiring of sufficient officers, technological support such as preliminary breath testing
devices and traffic cameras, and more), law enforcement agencies, command officers, and patrol
officers need to understand the importance of traffic safety laws and then take action to support
and enforce them. Consistent enforcement of these laws can help provide a safe and healthy
environment. At times, enforcement also must include “high visibility” enforcement. This will
support any messaging being done. It is through a high visibility fashion that the citizens begin to
see and understand that violating the law can have consequences and change their behaviours.

Additionally, when speaking of enforcing traffic laws, it is more than being caught violating the law.
The traffic stop is critical, but it is also important the offender face consequences from the
violation. This requires that the courts effectively process cases. Prosecutors (if they are involved
in the cases) and judges/magistrates must understand the importance of traffic enforcement. A
court is actually a system, which requires each component, police, prosecutors, and judges,\(^5\) to be
actively involved.

Using the objectives expressed in the Education & Training Pillars (section 3), the DBET
Subcommittee will promote effective traffic safety training of law enforcement agencies, command
officers and patrol officers, as well as for prosecutors and judges when appropriate.

**Law Enforcement Corruption**

As mentioned, some law enforcement officers look at traffic safety as a tangential requirement of
“protect and serve,” however, another important issue is corruption. Police corruption is an
international problem, and while not all officers are corrupt, the perception in some countries is
that all law enforcement officers are corrupt and that a bribe will get the offender out of any
violation.

Effective traffic safety efforts may require that corruption is addressed in a comprehensive
approach. The perception and the reality of corruption undermine any effort to establish the rule of
law as paramount. Without the support of the law enforcement in upholding the law and the
understanding by the public that the laws apply to all, there will be no change in behaviour.

Thus the elimination of any law enforcement corruption should be considered in any part of training
and/or reform effort. This will require an analysis of the communities where traffic safety efforts
are progressing. It will also require the development of an independent oversight
agency/committee to monitor the changes and enforce the reform. Keeping the public informed
about the efforts and transformations that are occurring will bring about support by the public.

---

\(^4\) Globally, the World Health Organization (WHO) has indicated that road deaths is 9\(^{th}\) as a cause of death in the world.

\(^5\) In a few jurisdictions supervision officers or probation officers may also be involved.
Dealing with traffic offenders

Traditional methods used to manage traffic offenders have used monetary fines and driver license sanctions (suspension, disqualification) as the principal means to penalise illegal behaviour.

Many jurisdictions have also introduced demerit point schemes that are managed through driver licensing administration.

Over the past two decades, some jurisdictions have legislated a requirement for driving offenders to attend a traffic intervention program before re-licensing.

Additional means of dealing with offenders include vehicle sanctions (impoundment) or requirements to fit intervention devices (such as alcohol ignition interlocks for alcohol-impaired drivers).

The DBET Subcommittee will identify and promote effective approaches that can be used to address driver offending behaviour. The Subcommittee notes that the behaviour of novice drivers under supervision by parents or driving instructors within a Graduated Driver Licensing System (GDLS) is relatively offence-free, and that it appears that the move to solo (unaccompanied) driving, often with per passengers, is associated with the unlearning of safer, less risky driving behaviour.
Endorsements

The International Road Federation (IRF) is a not-for-profit, non-political, organization with the mission to encourage and promote development and maintenance of better, safer and more sustainable roads and road networks. Endorsing specific traffic safety concepts is one way to publicly demonstrate support for specific traffic safety activities and encourage a change in policy.

When appropriate, the DBET Subcommittee will develop endorsements of traffic safety actions that are under the DBET Subcommittee’s purview and submit them for consideration by the Road Safety Committee and the IRF.
TOPIC: SUSTAINABLE TRANSPORT

THE IMPACT OF HIGH OCCUPANCY VEHICLE POLICY ON TRAFFIC PERFORMANCE OF DR. DJUNDJUNAN STREET IN BANDUNG INDONESIA

Yohanes Samuel

Student ID: 2011410093

Civil Engineering Department

Parahyangan Catholic University, Bandung, Indonesia

2014
ABSTRACT

Dr. Djundjunan street is an arterial road in Bandung and located after Pasteur Toll Gate. This street usually experiences the domino effect from vehicles queue at Pasteur Toll Gate, especially during weekend. Traffic congestion on Dr. Djundjunan street cannot be avoided, so that Bandung Municipality establish high occupancy vehicle named “4 in 1”. But the impact of this policy is traffic congestion on Prof Dr. Surya Sumantri street which is located parallel street to Dr. Djundjunan street. In order to provide solution of the problem, evaluation of this policy is crucial.

Keywords: High occupancy vehicle, “4 in 1” policy, Dr. Djundjunan street, Prof. Dr. Surya Sumantri street, Bandung Municipality.

INTRODUCTION

Indonesia is a developing country with high population growth rate. Badan Kependudukan dan Keluarga Berencana Nasional (National Family Planning and Population) noted that the growth rate is increasing 1.49% per year (BKKBN, 2013). This condition triggers a rapid increase in the number of vehicles. Central Bureau of Statistics noted that in 2012, the number of vehicles has reached 94,373,324unit and the growth of number of vehicles has reached 10.25% (Central Bureau of Statistics, 2013).

Bandung the capital of West Java province is the fourth most populous city in Indonesia. It located near Jakarta the capital city of Republic of Indonesia, so that Bandung is a strategic location for economy, business, education, and culture. In the year 2013, vehicles growth rate in
Bandung is 11% per year (Head of Traffic Police Corps of the Republic of Indonesia, 2014). For the distribution of transportation modes, people prefer to use their private vehicles, which are motorcycles (55.78%) and private cars (30.96%) rather than use public transports (13.26%). Bandung Transportation Agency (DisHub Bandung) had noted that last 5 years, the road network growth is only 1.03%. The total scale of road networks area in 2011 was 2.96% of the total area in Bandung. Ideally, 10% to 30% of the total area in Bandung is supposed to be functioned as road networks (Bandung Transportation Agency, 2011).

During weekends, Bandung is always crowded by domestic tourists, especially from Jakarta. They usually shopping and held a family gathering in Bandung. Because of that, Pasteur Toll Gate, as one of inner gates to Bandung is usually busy. In 2013, The number of vehicles that enter Bandung through Pasteur Toll Gate was 14,128 unit vehicles per day during weekend (Kurniawan, 2013) and increased up to 36,000 unit vehicles during long weekend (Mulyawati, 2012). Dr. Djundjunan street as an arterial road in Bandung and located after Pasteur Toll Gate usually experiences the domino effect from queue vehicles at Pasteur Toll Gate.

Bandung Transportation Agency (DisHub Bandung) noted that during peak hours on the weekend, the number of vehicles enter Dr. Djundjunan street from Pasteur Toll Gate is 10,011 vehicles per hour which is greater than the capacity of Jalan Dr. Djundjunan (7,529 vehicles per hour). Bandung Transportation Agency (DisHub Bandung) implemented high occupancy vehicle named “4 in 1” every Saturday from 09.00 am until 01.00 pm (Soedrajat, 2013) according to Bandung mayor's actNo.551/Kep.582-DisHub/2013 about determination of area traffic control and liability of at least four persons in one private car on certain roads in the city.
The objective of the policy is to aim the reduction of number of vehicles that pass through Dr. Djundjunan Road but can increase the number of passengers. But many people think that this policy is useless, ineffective, and give a negative effect for other adjacent roads especially on Prof. Dr. Surya Sumantri street (Baso, 2013). Therefore, evaluation of this policy is crucial. The research objectives are analyzing the traffic performance of Dr. Djundunan street due to the implementation of “4 in 1” policy and then evaluate the impact of the policy on Prof Dr. Surya Sumantri street as a parallel street to Dr. Djundunan street.

**CONCEPT OF HIGH OCCUPANCY VEHICLE**

In United States of America, the implementations of high occupancy vehicle (HOV) system concept have been proven to be flexible and cost effective alternatives in increasing the capability of congested urban transportation systems to move people. HOV facilities are an effective means of moving people; they encourage significant numbers of commuters to choose to ride a bus, vanpool, or carpool for reach their destination.

*Graphic showing that one bus, or six vanpools, or 15 3+ carpools, or 22 2+ carpools, or 45 single-occupant vehicles, are necessary to transport 45 people. (U.S. Department of Transportation, 2012)*
An intent of all HOV systems implementations is to help maximize the number of persons moved on a roadway by increasing the average number of persons per vehicle, so effectively doubles the capacity of the roadway to move people. In U.S., developing a high-occupancy vehicle project typically involves designating a special roadway or lane(s) that is reserved for exclusive use by high-occupancy vehicles during at least portions of the day. But in Indonesia, especially in Bandung, Bandung Municipality has a crucial issue with land use so Bandung Municipality can't build or designating an additional special roadway or lane(s) on the existing roads. In Dr.Djundjunan street, Bandung Municipality has implemented high-occupancy vehicle named as “4 in 1” that only allow vehicles with four people in a car can pass Dr. Djundjunan street. Every Saturday at 09.00 am until 01.00 pm has always been the peak hour for car drivers who want to enter Dr. Djundjunan street from Pasteur Toll Gate. Because of that, “4 in 1” policy is applied only at that time, once in a week.

High-occupancy vehicles projects can’t be applicable everywhere. A safe and cost-effective travel is few of the reasons. The reasons why high-occupancy vehicle system project is the right choice to be applied in Dr. Djundjunan Road are listed below.

- *Increase the Average Number of Persons per Vehicle.* High occupancy vehicle are designed to get single-occupant drivers choose to use public transportation. HOV on Dr. Djundjunan street provide to move more people in less vehicle amount, especially during weekend.

- *Preserve the Person-Movement Capacity of the Roadway.* A single high-occupancy vehicle lane assures that capacity will be available in the future to serve growth in person travel. The HOV lane effectively doubles the capacity of the roadway to move people.
- **Enhance Bus Transit Operations.** Because of HOV, more commuters will attract to use public transports especially bus. But in Bandung, so far there isn’t any public transport that can be trusted or can be relied on because lots of issue and problem about Bandung’s public transports still occur.

- **Capital Costs.** High-occupancy vehicles facilities are relatively inexpensive, so this is the best choice for the city of Bandung limited funding.

- **Public Operating Cost are Low.** Because of the HOV policy, one car that passes through Dr. Djundjunan Road contains at least four persons. This car trips are served at a very low marginal public costs, because HOV policy makes the car operates more effective. As a result, total public operating cost per passenger on HOV facilities is low.

When a decision is made by Bandung Municipality to apply HOV as a solution for answering the congestion problem, other considerations that enhance high-occupancy vehicles success should be considered. The considerations are listed below.

- High-occupancy vehicle lanes should be implemented as new lanes. Conceptually, an HOV lane can be created either by adding a new lane to a facility, which is then designated as an HOV lane, or by taking a lane away from general-purpose traffic and designating it as an HOV lane. But at the top of this essay, I have said that Bandung has a crucial issue about land-use so at Dr. Djundjunan Road, created a new lane only for HOV lane is an impossible thing. So the local government had already agreed that all of existing lanes leading into the city of Bandung at Dr. Djunjunan Road are applied with HOV policy, this policy is known as “4 in 1” policy.
• High-occupancy vehicle lanes involve a system improvement. In addition to providing an exclusive lane for use by high occupancy vehicles, successful implementation of an HOV project generally involves providing a system of improvements. Some of these complementary system improvements involve construction of physical facilities, such as bus transfer centers, park-and-ride lots, and HOV bypass ramps. In Dr. Djundjunan Road, this action hasn’t been implemented. So great hope that local government can improve the HOV system with adding additional facilities that will help the HOV policy to get its goals. The success and acceptance of an HOV project can be highly dependent on pursuing the appropriate package of complementary actions and strategies. Simply constructing or designating a roadway lane as a priority HOV lane does not assure that the project will success.

• High-occupancy vehicle projects often involve multiple agencies. Transit agencies are frequently responsible for providing some of the support facilities and services that are needed to maximize HOV lane effectiveness. For example; because of Dr. Djundjunan Road is applied with “4 in 1” policy, personal commuters who need to travel through Dr. Djundjunan Road can use bus transit facility or another transit facilities, so they can pass through Dr. Djundjunan Road. If they bring their personal car, they can park their car at parking lots that some other agencies provide. So in this case there are two additional agencies that involve at this situation. These agencies are needed to make “4 in 1” policy work. But unfortunately, it is not yet implemented properly by the local government. Decisions have to be made regarding agency participation for funding of capital and operating costs.
DATA COLLECTION

Primary data and secondary data is needed in this study. Primary data is carried out several times on Dr. Djundjunan street and Prof. Dr. Surya Sumantri street at the same time in Bandung during 09.00 am until 01.00 pm in 18th October, 25th October, 1st November, and 8th November 2014. The field data are traffic volume per hour and vehicle speed per hour. Secondary data is obtained from Bandung Transportation Agency (DisHub Bandung). The secondary data are vehicle speed per hour and traffic volume per hour on Dr. Djundjunan street and Prof. Dr. Surya Sumantri street in 2011 when the “4 in 1” policy had not been established by Bandung Transportation Agency (DisHub Bandung).

DATA ANALYSIS

Comparing the primary data and the secondary data about vehicle speed per hour and traffic volume per hour at Jalan Dr. Djundjunan and at Jalan Prof. Dr. Surya Sumantri using a statistical hypothesis test, it can be called as “t test”. This test is used for finding the best conclusion that we can accept about whether “4 in 1” policy is the right decision to answer the congestion problem or not. If the answer is not, another option must be made so this problem can be solved. From the primary data and the secondary data, degree of saturation value can be calculated. The degree of saturation value which has been calculated, can be compared using a statistical hypothesis test.
To calculate degree of saturation:

\[ DS = \frac{Q}{C} \]

with: \[ DS \quad = \text{Degree of saturation} \]

\[ Q \quad = \text{Traffic flow} \]

\[ C \quad = \text{Road capacity} \]


Pooled Standard Deviation:

\[ SP = \sqrt{\frac{(n_1-1)\times S_1^2 + (n_2-1)\times S_2^2}{n_1+n_2-2}} \]

with: \[ SP \quad = \text{Pooled standard deviation} \]

\[ n_1 \quad = \text{Total data from the first sample} \]

\[ n_2 \quad = \text{Total data from the second sample} \]

\[ S_1 \quad = \text{Standard deviation of first sample} \]

\[ S_2 \quad = \text{Standard deviation of second sample} \]
Statistical Test:

\[ t = \frac{(\bar{y}_1 - \bar{y}_2)}{\sqrt{\frac{s_1^2}{n_1} + \frac{s_2^2}{n_2}}} \]

with: \( \bar{y}_1 \) = Mean of first sample

\( \bar{y}_2 \) = Mean of second sample

Reference from: An Introduction To Statistical Methods And Data Analysis. 4\(^{th}\) ed. (Ott, 1993).

**CONCLUSIONS**

Still lots of road user especially drivers, who driving pass through Dr. Djundjunan Road from Pasteur Toll Gate, and several local government parties think that this policy is useless and ineffective; even give a negative effect for other roads adjacent especially Prof. Dr. Surya Sumantri Road. If the implementation of HOV projects in Jalan Dr. Djundjunan isn’t on a right target, we can agree with the negative thoughts about that policy. So the analysis about this policy must be done to produce the best conclusions and the best solutions for this policy. Lots of advantage in the implementation of HOV projects, such as high-occupancy vehicle lanes can move large numbers of people with fewer vehicles, high-occupancy vehicles system is inexpensive so this policy is the right answer for Bandung because of Bandung’s limited funding, increasing the number of persons per vehicle and reduces the rate increase in vehicle-miles of travel, which lessens transportation energy consumption. But the implementation of this
project can create congestion on Prof Dr. Surya Sumantri street as a parallel street to Dr. Djundunan street.

Implementation of this policy is still not perfect. Still lots of work must be done by Bandung Municipality so the this policy can achieve its goal and can bring the best result. If Bandung Municipality wants to improve the performance of “4 in 1” policy, than first thing Bandung Municipality should do is improving the public transportation facilities especially on Dr. Djundjunan street. With the policy move along with the good public transportation facilities, it can be confirmed that the traffic volume on Dr. Djundjunan street will decrease.

REFERENCES


TOPIC: ROAD SAFETY

THE MINIMUM SAFETY SERVICE STANDARD
ON PADALARANG-CILEUNYI TOLL ROAD

Dessy Dwi Ros Aninda

Student ID: 2010410133

Civil Engineering Department
Parahyangan Catholic University, Bandung, Indonesia

2014
ABSTRACT

Padalarang - Cileunyi toll road is one of the toll roads in West Java, Indonesia. It crosses Bandung City, Bandung Regency, West Bandung Regency, and Cimahi City. Padalarang - Cileunyi toll road accommodate high traffic and as alternative way to decrease traffic jam. In order to minimize traffic problems, Padalarang - Cileunyi toll road has to fulfill minimum service standards. The substance of toll road safety service indicators are traffic sign, road markings, guide posts, stakes km per 1km, right of way, and handling accidents. The purpose of this study is to evaluate performance of Padalarang-Cileunyi toll road towards fulfillment of minimum service standards of toll road in order to maintain operational effectiveness. This study will be beneficial for other toll road in Indonesia to fulfill the minimum safety standard.

Keywords: toll road, road safety, minimum service standards of toll road.

INTRODUCTION

Indonesia is a developing country that still developing all aspects, such as economy, education, politic, social, culture, and infrastructure. West Java Province is one of the 34 provinces in Indonesia with high population growth and rapid development in infrastructure including transportation facilities. Transportation is the movement of goods and people from one place to another that usually mention from origin to destination. It gives benefits to the community life in terms of economic, social, and politic.
From the economic point of view, transportation helps moving goods from the producing to the society easier in order to fulfill the society needs. In terms of politic, transportation plays an important role for an archipelago country like Indonesia. Creates unity and national unity between its people, makes them grow stronger as well as develops community services to a more evenly on every part of Indonesian territory.

**ACT NO. 38 IN 2004 IN INDONESIA**

According to the Act No. 38 in 2004 in Indonesia, the road is the land transportation infrastructure that includes all parts of the road, including complementary buildings and equipment intended for traffic, which is at ground level, above ground, below ground and/or water, as well as on the water surface, except railroad, lorry road, and the cable.

The road parts include the road benefits space, road area and supervision of road space. The road benefit space covers the road, the curb line, and the safety threshold. Road area covers space and the benefits of certain downstream land beyond the benefits of the road while the supervision of the road space is a certain space outside the road area that is under the control of the organizers.

Traffic sign is one of the equipment that can be either symbols, letters, numbers, sentences or a combination of them that serves as a warning, ban, orders or instructions to road users. As a means to control traffic, especially to improve the safety and smoothness of the road system, the bullet made / installed markings and traffic signs that can deliver information (orders, prohibitions, warnings, and instructions) to road user, and can affect road users.
In order for a sign / markings to be effective, it must meet the following requirements: have a specific needed, can be seen clearly, full of attention, have a clear and simple purpose, syntax fully respected and adhered to by the road user, provide sufficient time to respond. In order to have clear view for road users to see the sign, sign letter should contrast with the background, no obstructions such as plants or other overlap signs, located at an adequate distance regarding speed limit, and can be seen in the dark.

**TOLL ROAD**

According to the function, the road consists of public roads and special roads. Public road is a road reserved for common traffic, while the special is the roads built by the agency, individual enterprises, or community groups for their own interests. The government has authority to hold the road. Implementation of the road, as one part of the organizations of the transportation infrastructure, involved elements from society and government.

In order to provide optimum service, an integrated road sectors for example government, society, and business have to work together. Indonesia government build national road with status of toll roads wherein road users are required to pay a toll (Government Regulation of the Republic of Indonesia, 2005). This specification is intended to accommodate the potential for high traffic and alternative way to decrease traffic jam. The toll road is expected to accelerate economic growth and improve the welfare of people's lives in order to move more quickly, easily and safely.
A toll road is built with a consideration of the safety, security, and convenience of road users. The security factor can be realized with the release of a design area of criminal acts. While safety factor can be realized by the standard design of highway required including road geometric design, road furniture design, and road pavement design. Furthermore, comparison between the level of customer satisfaction and level of service quality is needed.

**PADALARANG - CILEUNYI TOLL ROAD**

Padalarang - Cileunyi toll road is one of the toll roads in West Java that crosses Bandung City, Bandung Regency, West Bandung Regency, and Cimahi City. The Padalarang – Cileunyi toll road operated in 1991 with a length of 58.5 km and is managed by PT Jasa Marga (Persero) Tbk. The environment around the toll road is mostly farming area, residential area, and industrial area.

Padalarang-Cileunyi toll road is expected to have capacity and level of service better than those that not toll roads. Padalarang – Cileunyi toll road have 8 toll gates with a range of number of booths from 2 to 13.

The availability of NSPM (Norms, Standards, Guidelines, and manuals) related to the operation of toll road, are important references for good service infrastructure quality and toll roads facilities. There are minimum services standards consist of measurement in the implementation of toll road management.

Government Regulation no. 14/2004, Article 8 controlled substance service includes 4 things i.e. the condition of the toll road, average travel speed, accessibility, mobility and safety. While the Government Regulation No.
392/2005, added the substance regarding service unit for help/rescue and relief services.

In order to implement the rules and indicators values contained in the Minimum Service Terms (MST) the toll road still requires monitoring system that includes policies and procedures for monitoring the toll road concession in accordance with the MST. The implementation of this monitoring requires additional tools such as toll road operation mapping tool, which is equipped with available facilities, as well as a variety of operations, which covers traffic characteristics, traffic accidents, or road environment that is intended to support the monitoring system.

According to regulation of the Minister of Public Works No. 392/PRT/M/2005 regarding the Minimum Service Standards of Toll Road, routine and intensive monitoring system on Padalarang-Cileunyi toll road section is needed in order to provide the service. Traffic accidents are a major indicator of the road safety level. Traffic accidents cause much greater loss, both lives and money. Traffic accidents can be caused by several factors i.e. human factors, vehicle factors, road geometric factors, wrong installation of traffic signs, and road pavement factors. Accident rate is a quantitative measure or scale to describe safety condition of a road segment.

According to Minister of Public Works Act No. 392/PRT/M/2005 this study evaluate the performance of the Padalarang-Cileunyi toll roads, by examining toll
road safety indicators i.e. signs, traffic regulation, road markings, and right of way. 

For all toll roads in operation, the indicators of minimum service standards for ruggedness is a maximum five years of 5 years to fulfill, and right of way fence indicator is a maximum period of 3 years with the implementation done in stages (Minister of Public Works No. 392 Section 8, 2005).

CONCLUSIONS

The purpose of this study is to determine the suitability of safety services towards fulfillment of the Toll Road Minimum Service Standards in order to maintain an effective operation of the toll road. Padalarang-Cileunyi toll road in West Java Province in Indonesia is the case study. Road safety service indicators used are traffic sign, road markings, guide posts, stakes km per 1km, right of way, and handling accidents. Related regulations, standards, manuals, and research reports regarding safety of toll roads in Indonesia used as references. This study will be beneficial to other toll road s in Indonesia or other toll roads in developing country with similar traffic and geometric conditions.

BIBLIOGRAPHY


**ABSTRACT**

The paper presents the singularity of Italian cities and of their historical centers. In these morphology and dimension together with the presence of the vast historical heritage leads to the use of historical buildings for public and representative functions making the historical centers powerful attractors. This typical Italian reality has caused high mobility demand which contrasts with the limited physical space at disposal due to the morphology of the historical centers. This situation makes it difficult to implement traditional patterns for vulnerable road users safety. The paper gives an overview of the urban policies applied in Italy to overcome the safety and quality issues set by Italian cities in time. It also offers a deeper insight to the problem through the case study of the city of Brescia.
Public spaces in Historic Cities and Vulnerable Users Movement

Michèle Pezzagno, Giulio Maternini
University of Brescia, Department of Civil engineering Architecture, Land, Environment and Mathematics
e-mail for correspondence: michele.pezzagno@unibs.it

The attention toward the topic of the historic centers in Italy progressively grew during the XXth century, together with its definition from the town planning point of view\(^1\). As a matter of fact, it is during that period that the first debates started about the renovation/preserving processes of the ancient part of the cities.

Europe registers the highest density of historic centers in the world: in particular, in Italy historic centers are more than 22,000. They all date back to different ages (Etruscan, Greek, roman, medieval, renaissance, baroque, fascist) and have different sizes. Some of them are very small and look like ancient boroughs (like San Giminiano, Volterra, Sirmione, etc.); some others are known as “minor historic centers”\(^2\) (like Brescia, Cremona, Lucca, Verona, Siena etc.); some others are middle-large sized historical centers (Genova, Venezia, Firenze, Roma, Milano, etc.).

In most cases, historic centers are characterized by the presence of consolidated urban functions, which are the result of a complex cultural, urban and socio-economical process and are also important traffic generators.

In Italy, the topic of a safe mobility in historic centers is considered an important issue in the academic debate (especially in terms of accessibility, use and conservation of the public spaces), but there still are some objective difficulties in introducing effective actions aimed at improving the vulnerable users movement, in particular pedestrians and cyclists. The reason can be found not only in the morphology which characterizes the ancient city cores (narrow roads, high urban density), but also in the different qualities of the historical centers which are evident in relation to the different phenomenologies present within the same historical centers. Such complexities can be summarized in macro-themes as explained below:

- Historic urban areas in which the relationship between the city and the territories (Choay 1988; 1992) has been mediated and then consumed by subsequent expansions. Such use led to a strong functional turnover. This is the case of the historic centers in the major Italian cities, such as Firenze, Milano, Roma, etc., where significant actions aimed at improving their livability, through the implementation of integrated urban policies and through the introduction of good quality public transportation systems were taken. In Italy there has been a significant delay in introducing efficient high quality public transport systems due to the complexity of historical centers which led to increasingly high and not easily quantifiable costs. Anyhow in big cities integrated policies addressed to the improvement of the inhabitants livability were gradually introduced in city centers. Such policies were finalized on the one hand to the improvement of the accessibility for residents and for the city users (in particular of tourists), on the other to the preservation of the functional mix (residential, commercial, administrative, services) as strategy to preserve the identity of the historical public spaces. These actions consisted in significant public investments aimed at the building refurbishing also at an urban scale using incentives and, at the same time, at the improvement of the Local Public Transport network, - trying to introduce efficient intermodal nodes - pedestrianizations of roads/squares - maintaining the attractiveness of the commercial axes – through traffic limitation and road pricing policies in order to favor daily use of public space for residents.

---

\(^1\) In urban planning, historic centres are considered homogeneous areas (or azones) and are identified with the letter A (ref. to rif. art. 2, c. 1, of the Interministerial Decree n. 1444, April 2\textsuperscript{nd} 1968). These zones include urban agglomerations which are characterized by significant historic, artistic, environmental features.

\(^2\) “Minor historic centres” are usually scarcely known, but play a relevant role in the structural identification of the territories where they are located.
• Minor historic centers the fulcrum of the surrounding territories throughout time from an environmental, morphologic, landscape and functional viewpoint.

• In most cases, their value is not properly recognized, and they suffer from the progressive prevailing of consumerism standardized models (offer/demand) which often are in contrast with local tradition and culture (Virilio, 1984). Local traditions, culture and economy haven’t got the energy to contrast the new functional models: trading and retail in particular. In this case urban policies are divided between the necessity of maintaining the suburban areas accessibility of the historical center and the technical and economical impossibility of the local public administration to pay for high quality efficient public transport system due to the city dimension.

This is the reason why the most part of the implemented policies mainly consist in the institution of limited traffic zones together with parking structures on the city core borders. As a consequence, the lack of competitive alternative access modes keeps the individual motorized traffic demand very high by residents and city users because of the presence of fundamental functions set exclusively in buildings in the core of the cities. Alongside this, in minor urban centers it is very hard to modify the existing spaces to introduce exclusive areas for pedestrians. In time “prevailing pedestrian areas” were introduced. In such areas only authorized vehicles (for residents and for freights delivery), public transport vehicles (busses, trolleys, trams, etc.) and bicycles are allowed. In these areas the circulation of mopeds and scooters has been recently excluded.

Minor historic centers

Figure 1 - Bergamo

Figure 2 - Brescia
Small historic boroughs characterized by abandon phenomena or, on the contrary, by seasonal overcrowding problems. The structure of the city usually does not require public transport services (as the transport demand is generally weak and the number of inhabitants does not justify their introduction). In most cases it is possible to access the boundary of city cores using individual motorized means of transport, leaving the cars in parking areas just outside. The qualification of the pedestrian path becomes fundamental and should be characterized by good design including path geometry and dimension considering also its length. Historically the most important element in pedestrianization policies of ancient boroughs is represented by parking pricing policies able to favor a pleasant and safe pedestrian access to the core.

Small historic centers

Historic centers, especially in Italy, represent a “fixed capital” at communities disposal, but often they are underused. The reasons of their marginality are multifield. First of all, they suffered from the last sixty years socio-economic, urban and metropolitan conflicting dynamics, combined with the necessity of investing significant resources to improve quality and accessibility to the buildings with public functions located inside the historic centers.

In particular, where the historic centers are landmarks of cultural heritage – apart from their dimension - and therefore focal attractions from the touristic point of view (such as Venezia, Firenze but also the borough of San Giminiano), the issue of the pedestrian capacity in relation to the physical space available can be solved only limiting pedestrian accessibility to the most crowded spaces. Alongside this issue there is the one of safety in case of emergencies where evacuation planning actions are particularly difficult.

2. THE CASE OF THE HISTORIC CENTER OF BRESCIA

Brescia historic center\(^3\), which occupies an area of about 1 km\(^2\), is entirely contained in the perimeter of the ancient city walls and can be considered a “minor historic center”. It is characterized by a strong stratification of urban functions linked to the presence of several civic/administrative, commercial an religious activities and services which attract users both at local and territorial level (Tiboni eds, 2005). It hosts some institutional headquarters, some universities, social sanitary assistance services, several commercial arterials and touristic attractions. The city core preserved in time most of its urban centralities in terms of functions and identity-places (such as historic roads and squares), attracting significant traffic flows from the territories. Such spaces throughout time have remained generators and attractors of mobility in the urban complexity.

Contemporary Brescia lacks identity in its public spaces: the new squares are often empty urban spaces or mere mobility nodes and have lost their vital system of relationships between the human dimension and the functional buildings. Excluding the green areas, the public spaces dedicated to leisure activities are more and more intro-flexed, out of scale and not related to the territorial context. These “new functional spaces” are the consequence of a process of the physical agglomeration of the retail and entertainment functions. They are set in big malls and they become a leisure time attraction using the traffic nodes, the parking areas and the highways alongside the city. Because of their position, and accessibility they have become the main competitors of the social life in the historic center.

For what concerns the historical city polarizing spaces – public spaces rich in heritage - their function and design have remained valuable through time maintaining their original aim.

\(^3\) The city of Brescia, like all the European cities, experienced a complex process of urban development, characterized by the progressive stratification of urban expansions through time. It is possible to identify three significant stages of development: from the first human settlements which took place in the ancient era to the first historical walls demolition in the pre-modern age; the development which characterized the city during the modern age (from the early 1900s to the post-second world war period); the contemporary city (from the economic boom of the sixities to today).
In the city of Brescia there has been a strong link among design/function and way of using the public space made up by the three main historical squares set in the historical center: piazza del Duomo – which has both religious and civic functions – Piazza della Loggia – which has both civic and trading functions – and Piazza del Mercato – which has trading and recently civil function.

This articulated set of functions becomes a catalyzing element of mobility and it is served by integrated mobility. In the last twenty years, the mobility policies of Brescia concerning the access to the central urban areas have been oriented toward prevailing pedestrian or transit modes, though assuring the possibility to get close to the city center by means of individual modes.

The present collective transport modes offer consists in: one automated light metro line, recently activated (March 2013), which crosses the historical center North-Southward and has two stations (San Faustino and Vittoria) inside the city core and one station (Stazione FS) very close to it; an urban bus network which serves the roads all around the city core and its main inner roads; a bike sharing service, which has 11 stations inside the city center and just as many around it; a car sharing service that has parking areas inside the city center; a last mile freight distribution service also with e-bikes and e-vehicles.

Figure 5 – Functional and morphological structure of the Brescia historical center
Alongside, a railway station and an extra-urban bus station are located close to the South-Western corner of the city center. These stations represent important inter-modal nodes and serve the accessibility to the city center from the territories.

As far as the individual motorized mobility, the adopted policies were oriented towards forms of traffic limitations, together with the localization of structured parking all around the city center and a progressive pedestrianization of the most inner areas.

In the case of Brescia there is a great consciousness about the need of promoting urban policies which aim at historical center pedestrianization. Returning the historical center of Brescia to pedestrians is linked to the possibility of using the ancient squares as livable spaces—used by people to socialize and meet—, instead of parking areas for residents and city users. The historical center of Brescia is well structured for pedestrian movement, such public space system develops through porches, covered pedestrian spaces, squares among buildings used for public needs. The demise of pedestrian function implies the decline of most of the historic city but also of the contemporary one.

Through time the effectiveness of policies for the historical center has been backed by ambitious plans for the city as the introduction of innovative high quality transport systems (automated light metro line) and bike sharing.

It is important to remember that such actions in a middle size metropolitan city as Brescia are not easily carried out on both on the economical investment side and because of the urban context characteristics and dimension.
It is a complex and integrated planning action in which often high specialized projects are needed due to the presence of historical sites together with the underground archeological ruins (Roman and Pre-roman); compact blocks of buildings with high density of inhabitants; characterized by reduced space at disposal and preexistences that cannot be ignored - such as ancient important buildings and underground infrastructures and in particular the urban district heating.

The implementation of such strong and effective mobility policy is a fundamental step to maintain and renew the quality of urban historical public spaces used by non- motorized users – avoiding the abandonment of the historical center –. The existence in a city of a high quality integrated public transport system⁴, as for example the automatic light metro, sets the foundation for effective renewal urban policies in historical centers. Such policies should aim at maintaining and revaluing the original urban functions improving the quality of pedestrian paths network in relation to the metro-line stations taking into account the non-motorized flows related to them.

Figure 8 - Via Trieste in Brescia. The Level of service of the walkway to reach the Catholic University is E⁵.

Figure 9 – Via Mazzini in Brescia. The Level of service of the waiting area at the bus stop is E.

Brescia best practice is useful to understand that harmonized and integrated policies are the only way to give the historical center back to the citizens allowing them to move safely in lovable streets through the ancient buildings. The cost of 13km metro line in Brescia has been of about 950MLE. It has been a very high investment for the city but it has been made with the consciousness of the return of the investment not only on the infrastructure itself but also in relation to the existing building capital mainly represented by the artistic and historical heritage and its related representative functions.

⁴ The metro stations have been integrated with bus services, parking areas, bike sharing also thanks to mobility management actions co-funded by CIVITAS PLUS MODERN 2008-2012.
⁵ The level of service E corresponds to oversaturated pedestrian traffic levels. This situation pushes pedestrians to walk outside the walkway, on the carriageway where motorized traffic is present.
References (on Brescia)

A.A.V.V., (1980), "Il volto storico di Brescia", 5 Volumi, Grafo Edizioni, Brescia
A.A.V.V., (1981), "Brescia moderna - la formazione e la gestione urbanistica di una città industriale", Grafo Edizioni, Brescia
A.A.V.V., (1993), "La Loggia di Brescia e la sua piazza", Comune di Brescia, 3 Volumi, Fondazione CAB, Brescia
A.A.V.V., (1998), "Quaderno - Laboratorio Brescia PRG", Grafo Edizioni, Brescia
BUSI R, (1994) "Marcello Piacentini a Brescia: una presenza ineludibile", Quaderni di architettura e urbanistica dell'Università di Pisa, n. 3
ROBECCHI F., (1993), "Le strade di Brescia", Periodici locali Newton, Roma
PANAZZA, STRADIOTTI, (1980), "Brescia nelle vecchie fotografie", Grafica Gutenberg, Grafo Edizioni, Brescia
SINISTRI T., a cura di,(1981), "Brescia nelle stampe", Grafo Edizioni, Brescia
TIBONI M., a cura di, (2006), "La funzione delle piazze storiche oggi", Cescam Edizioni, Brescia
Overview

Rumble Strips are an effective countermeasure for preventing roadway crashes. The noise and vibration produced by rumble dots alert drivers when they leave the traveled way. In other country had four types of shoulder rumble strips. They differ primarily in how they are installed, their shape and size, and the amount of noise and vibration produced:

- Milled
- Rolled
- Formed
- Raised

In other Country, Toll Road agencies often use all types of shoulder rumble strips, depending on the need and the material. The groove pattern can be installed intermittently or continuously. The groove pattern, depth, width, shape, and spacing may also change with the road agency.

Marg MandaL HA Sakti PT

Marga Mandalasakti PT owns and operates Tangerang-Merak toll road, which is 72 km expressway with two mainlanes, serving the route due west from Jakarta. One of mained purpose journey of Tangerang Merak toll road users are headed to the port of Merak to cross the sea to island of Sumatera. One of the problems in the Tangerang Merak toll road is the high number of traffic accidents. For that, we want decrease number of the accidents, which applications the rumble
strips. But we modified the shape, width, thickness, and spacing, it called “Rumble Dots”.

**Purpose**

The main cause of roadway crashes is driver drowsiness and inattention, which are sometimes compounded by driving too fast. Alcohol and drugs can contribute to both fatigue and speed. Driver fatigue also is induced by Toll Road hypnosis, which occurs when the lines and stripes on long, monotonous stretches of Toll Road reduce the driver’s concentration. When drivers stray from the travel lane, rumble dot rouse their attention to allow a safe recovery. Rumble dot also are helpful in alerting drivers to the lane limits where conditions such as rain, fog, snow or dust reduce driver visibility.

Actually there are two main applications of rumble Dots:

- Centerline Rumble Dots – an effective countermeasure to prevent head-on collisions and opposite-direction sideswipes, often referred to as cross-over or cross-centerline crashes. Primarily used to warn drivers whose vehicles are crossing centerlines of two-lane, two-way roadways
- Shoulder Rumble Dots – an effective means of preventing run-off-the-road crashes. They are primarily used to warn drivers they have drifted from their lane. A variation on this is the edge line rumble dots, which places the pavement marking within the rumble dots, improving the visibility of the marking. This is more commonly used on roads with narrow shoulders.

From database caused of the accident in Tangerang - Merak Toll Road, we found that the cause of the accidents by human error, which one primarily is sleepy. And from those we got that the accident happened are mostly at shoulders.

Tangerang - Merak Toll Roads applied the rumble dots both sides of the street in order to decrease the number of the accidents at the shoulders.

**Graphics 1. Position of the accidents in 2010**

**Graphics 2. Caused of the traffics accidents in 2010**
The Implementations of Rumble Dots

What causes drivers to drive off the roadway or out of their lane?

Many factors and combinations of factors, contribute, including driver fatigue and drowsiness; distracted driving; and slippery road surfaces and poor visibility in adverse weather conditions. These factors are sometimes compounded by driving too fast. Alcohol and drugs can contribute to both fatigue and speed. Driver fatigue also is induced by Toll Road hypnosis, which occurs when long, monotonous stretches of Toll Road reduce the driver’s concentration (sleepy).

How do rumble dots prevent crashes?

For those drivers who are about to unintentionally drive off the pavement edge or cross the centerline, rumble dots create noise and vibration inside the vehicle through interaction with the vehicle tires. Often this alert is strong enough to get the attention of a distracted or drowsy driver, who can quickly make a corrective steering action to return to the roadway safely. Rumble dots also alert drivers to the lane limits when conditions such as rain.

Shoulder Rumble Dots are an effective means of preventing run-off-the-road crashes. They are primarily used to warn drivers when they have drifted from their lane.

Design Features – Placement of shoulder rumbles dots close to the road marking increases their effectiveness at intercepting and alerting a drifting motorist. Especially on narrow-shouldered roads, at the other country place the rumble dots at the shoulder edge, in conjunction with the edge line pavement marking, creating a “shoulder rumble dots.”

To accommodate road users, common modifications to shoulder rumble dots include 1) avoiding rumbles unless a minimum of 4 ft. of paved shoulder is available; 2) leaving a periodic gap in the rumble dot to allow road users to travel between the shoulder and travel lane; 3) modifying the dimension of the rumble to make the rumble dots safer for road users.
### Table 1. Number & Location of Rumble Dots Implementations

<table>
<thead>
<tr>
<th>No</th>
<th>Locations (KM)</th>
<th>Direction</th>
<th>Length (m²)</th>
<th>The Rumble Dots (units)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>41+300 - 45+400</td>
<td>A</td>
<td>4,100</td>
<td>20,500</td>
</tr>
<tr>
<td>2</td>
<td>50+100 - 52+900</td>
<td>A</td>
<td>2,800</td>
<td>14,000</td>
</tr>
<tr>
<td>3</td>
<td>53+100 - 56+400</td>
<td>A</td>
<td>3,300</td>
<td>16,500</td>
</tr>
<tr>
<td>4</td>
<td>77+735 - 83+045</td>
<td>A</td>
<td>5,310</td>
<td>26,550</td>
</tr>
<tr>
<td>5</td>
<td>83+045 - 85+000</td>
<td>A</td>
<td>1,955</td>
<td>9,775</td>
</tr>
<tr>
<td>6</td>
<td>95+399 - 93+032</td>
<td>B</td>
<td>2,367</td>
<td>11,835</td>
</tr>
<tr>
<td>7</td>
<td>70+720 - 59+339</td>
<td>B</td>
<td>11,381</td>
<td>56,905</td>
</tr>
<tr>
<td>8</td>
<td>41+015 - 38+887</td>
<td>B</td>
<td>2,128</td>
<td>10,640</td>
</tr>
<tr>
<td>9</td>
<td>83+045 - 85+000</td>
<td>B</td>
<td>1,955</td>
<td>9,775</td>
</tr>
</tbody>
</table>

### Materials specifications

1. Tack coat: tack coat medium curing 70 (MC 70) or rapid curing 250 (RC 250). The volume of the placing is 0,3-0,7 kg/m²

2. Hotmix

Hotmix manifold that we used is wearing(ACWC) with the requirements produced by mixing bitumen agencies (Asphalt Mixing Plant). Terms of hot mix properties as listed below.

### Table 2. Hotmix properties

<table>
<thead>
<tr>
<th>Properties of the mixture</th>
<th>Parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stability : kg, minimum</td>
<td>1.100</td>
</tr>
<tr>
<td>Flow : mm</td>
<td>2.5 - 4.0</td>
</tr>
<tr>
<td>Voids in total mix (%)</td>
<td>3.0 - 50</td>
</tr>
<tr>
<td>Voids Filled with asphalt (%)</td>
<td>7.5 - 85</td>
</tr>
<tr>
<td>Levels bitumen (%)</td>
<td>5 - 7</td>
</tr>
</tbody>
</table>
**Working methods**

1. First, we should clean up the pavement - flexible pavement- that we will put on the rumble dots using a compressor / blower.

2. After that, we coat the area by tack coat.

3. The final step is the placing.

The placing hot mix wearing a compacted material is divided into two ways; 1) the placing of material held in the area to be installed rumble dots, 2) the placing conducted outside the area (pre-press) for further spread in the area that will be installed rumble dots.

<table>
<thead>
<tr>
<th>Width</th>
<th>Length</th>
<th>Thickness</th>
<th>Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 cm</td>
<td>15 cm</td>
<td>0.5 cm</td>
<td>15 cm</td>
</tr>
</tbody>
</table>

Locations of implementations (KM)

83+000 - 85+000 A

Table 3. Dimensions & location of rumble dots implementation in 2010
<table>
<thead>
<tr>
<th>Year</th>
<th>Width</th>
<th>Length</th>
<th>Thickness</th>
<th>Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>2011</td>
<td>10 cm</td>
<td>10 cm</td>
<td>0,25 cm</td>
<td>15 cm</td>
</tr>
</tbody>
</table>

Locations of implementation (KM)

- 41+300 - 45+400 A
- 50+100 - 52+900 A
- 53+100 - 56+400 A
- 77+735 - 83+045 A
- 95+399 - 93+032 B
- 70+720 - 59+339 B
- 41+015 - 38+887 B

Table 4. Dimensions & location of rumble dots implementation in 2011

<table>
<thead>
<tr>
<th>Year</th>
<th>Width</th>
<th>Length</th>
<th>Thickness</th>
<th>Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>2012</td>
<td>5 cm</td>
<td>10 cm</td>
<td>1,5 cm</td>
<td>15 cm</td>
</tr>
</tbody>
</table>

Locations of implementations (KM)

- 60+000 - 68+000 B

Table 5. Dimensions & location of rumble dots implementation in 2012

**Result**

![Graphics 4. Number of accidents victims in 2010-2012](image-url)
Conclusion

1. From the result that we got, Rumble Dots applied at Tangerang – Merak Toll Road effectively decreased number of the accidents and loss of the assets. In 2010-2012, number of the accidents decreased 54.7% and the number of accidents victims decreased 47.5%.

2. The ideal dimensions of rumble dots

<table>
<thead>
<tr>
<th>Width</th>
<th>Length</th>
<th>Thickness</th>
<th>Jarak</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 cm</td>
<td>10 cm</td>
<td>1.5 cm</td>
<td>15 cm</td>
</tr>
</tbody>
</table>

Table 6. Standard dimensions of rumble dots

The dimensions we use as standard dimensions for implementations of rumble dots in other locations.
ABSTRACT:

Steel truss bridges cover about 10 percent of the total national bridging. During service time, overload truck, improper design and construction quality cause early deterioration of the concrete bridge deck. This was indicated by crack formation in the concrete slab of 19.3 percent of all steel truss bridges. Due to this condition, our institute was eager to implement a new deck system known as segmental orthotropic steel panel to replace the damaged concrete deck system in Citarum 1 Bridge in 2009. To indentify technical feasibility for application of this panel on other bridges, the level three of bridge monitoring was conducted in the form of field and laboratory testing. The field test results of the steel panel response show that some components are still in good condition and no visual damage was observed. However, during laboratory testing, some bolts connection of the steel panel have a tendency to lose tightness in some location in the panel especially in panel-to-panel connection. This identifies that attention is needed to maintain this steel panel system. Further study is also essential for stiffening the steel panel system movements in the bridge longitudinal direction.

INTRODUCTION

Steel truss bridges cover about 10 percent of the total national bridging. During service time, overload truck, improper design and construction quality cause early deterioration of the concrete bridge deck. This was indicated by crack formation in the concrete slab of 19.3 percent of all steel truss bridges based on IBMS (Interurban Bridge Management System) data.

Development of bridge deck construction techniques has enabled a system of segmental shaped panels made of steel having orthotropic properties for replacing the damaged in concrete bridge deck (Irawan et al 2010). Orthotropic properties obtained by the ribs that are welded beneath the floor plates with specific configuration to achieve greater stiffness compared to the bridge deck. The panels were each other connected by using bolt. Afterwards each segment connector plate was justified on the top panel and bolt connection system. However, to adjust the position and shape of the affected transverse bridge girder due to the existing vertical bridge alignment in the field, a gap was provided to allow movement in the longitudinal direction of the bridge.

The panel was not exactly the same as the patent of Korea and the United States with a variety of joint and connection type and not clearly intended for the replacement of the damaged slab of an existing steel truss bridges.

Segmental-orthotropic-steel-panel is meant to have a concept similar to the application of segmental concrete precast prestressed panels that have been developed and / or patented in Indonesia, with advantages in terms of reduction of dead load alone. This becomes quite important especially for minimizing additional handling fee for structural addition structures and the overall bridge structure.

However, that does not mean the system does not have a weakness Orthotropic bridge deck in the Netherlands and Japan have damaged structural elements in any relationship weld as seen in long-span bridges with the same structural floor system and have continuous spans as explained by Kolstein (2007), de
Jong (2006), and Kodama (2008). Based on the segmental panels that have been fully trialed out the field which explained by Irawan et al. (2009) the result of a one year monitoring of structural response to both in field and laboratory will be presented in following article.

METHODS

Field monitoring was carried out by using the concept of level 3 by Wenzel (2009), which is focused on the utilization of a reasonable number sensors that is sufficient to detect the structural performance. The sensors are placed in location to measure strain basen on strain-gauge and equipment to measure height differential between segmental panels due to traffic load. The data was recorded on the bridge for 3 x 24 hours. The amount of data that is quite a lot and therefore quantity identification of strain response is analyzed using rainfall counting method according to ASTM E1049 - 85. The location of sensors to measure the static response of structural elements can be seen earlier in Figure 1, described by Irawan et al (2010).

Testing of dynamic free and forced vibration due to load trucks with a total load of 217.6 kN was compared with similar testing results that was previously tested after completion of the prototype late 2009.

In addition the fatigue performance was assessed by cyclic load testing on the truck floor system simultaneously and dynamically with a frequency of 1 Hz up to 100,000 cycles. The expected result of this test is an accelerated damage to the structure model. Load placement is shown in Figure 2, with a total weight of 500 kN to represent of a truck load that is significant enough to cause damage and loss of firmness on the connection bolts on the bridge. Strain sensors based on strain-gauge as many as 31 pieces, as shown in Figure 2 were placed at the center and the part that is close to the bolt connection. While the displacement sensors on the model of the structure are 11 in number placed on the bottom of the cross girders and at the bottom of segmental panels, as described by PT Dirgantara Indonesia (2010). Afterwards some monitoring result were compared with the existing limitations from the technical provisions required in the design of steel structures for bridges in Indonesia.

(a) location in elevation (above) and cross-section of the bridge (below)
(b) location in elevation of the bridge

Figure 1. Sensor location on a) segmental-orthotropic-steel-plate and b) steel truss

Figure 2. Testing model for cyclic loading with numbered panel

And finally a comparison of visual inspection after nearly four years of construction reveals a wide variety of shape imperfections and loss of the elements of the existing bridge.

RESULTS

Furthermore sensors - sensors placed at a predetermined position produces strain response of steel frame as shown in Table 1. Strain that occurs is still under strain permission element or steel by 761 microstrain at 160 MPa according to the applicable specifications. By referring to the results in Figure 3, steel frame elements are still in a good condition.
Moreover, the deflection of segmental panel as shown in Figure 3 is still below 5 mm, which is the value of theoretical safe value of panel model. Also the strain that occurs in orthotropic plate was smaller than 761 microstrains prove that the panel is in good condition.

(a) strain in bridge truss bottom chord and diagonal chord

(b) deflection in segmental panel
The value of the bridge dynamic response, as can be seen in Table 1, particularly frequency value show that no change in value occurs. While the changes are significant in dynamic amplitude. And the value is slightly increased in damping ratio. The increase in the value of the bridge dynamic response show that there was an increase in stiffness in the structure due to confinement in the transverse direction of the bridge elements that have not allow to be installed in 2009.

Furthermore segmental panel in the laboratory system show that in some places, particularly the loosen bolt in panel-to-panel connection occur. This is indicated by the occurrence of the damage on the bolt thread, which is explained by PT Dirgantara Indonesia (2010) in accordance with the expected mechanism for prying force by Bickford (2008), as can be seen in Figure 5. However, in general no structural damage found in all plate elements of the segmental panel. It shown at the value of the maximum strain of 200 microstrain far below the value of 761 microstrains.

Table 1. Bridge dynamic response

<table>
<thead>
<tr>
<th>Condition</th>
<th>Frequency (Hz)</th>
<th>Dynamic Amplitude (mm)</th>
<th>Damping ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2009 First Operational</td>
<td>2.68600</td>
<td>1.55200</td>
<td>0.892</td>
</tr>
<tr>
<td>2010 Monitoring</td>
<td>2.68555</td>
<td>0.22414</td>
<td>0.99</td>
</tr>
</tbody>
</table>
Figure 5. Position of loosen bolt in testing model direction (symbolized by dotted and numbered panel based on Figure 2)

And finally through visual inspection, a pattern of cracks in the asphalt was found. This phenomena shows that an initial indication of the movement is quite big enough at the bottom of the asphalt layer. Referring to the result of laboratory testing, the observation, as shown in Figure 6, prove that certain stiffness in panel-to-panel connection was not enough to accommodate movement due to deflection at the segmental panel which caused by the gap in the panel to panel connection.
CONCLUSION

This study was conducted to investigate the performance of construction since the replacement of damaged concrete bridge deck using steel segmental panel. A serial of field and laboratory testing, and visual inspection after nearly four years of construction is completed. The results can be used as a reference for the refinement and improvement of the system as a good type to apply to the replacement of old steel truss bridge flooring damaged or as a new steel deck with superior reduction in dead load of the bridge.

Based on the results of the testing and inspection, it is indicated that there has been a loss of bolt connection tightness, especially on the joints between panels in the transverse direction of the bridge. By indicating that there should be an improvement efforts to strengthen the connection between the panel system by inserting a connecting element.

ACKNOWLEDGMENTS

This work was supported by Agency for R& D Institute of Road Engineering, Ministry of Public Works, Indonesia.

REFERENCES


**PAPER TITLE**
(90 Characters Max)
Introduction and Effects of Environmental Road Pricing Scheme where Inducing the Traffic of Urban Expressway to Parallel Route by Discounting the Toll Fare

<table>
<thead>
<tr>
<th>TRACK</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>AUTHOR (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Teruhisa ISHIBASHI</td>
<td>Assistant Manager</td>
<td>Hanshin Expressway Company Limited</td>
<td>JAPAN</td>
</tr>
<tr>
<td>CO-AUTHOR(S) (Capitalize Family Name)</td>
<td>POSITION</td>
<td>ORGANIZATION</td>
<td>COUNTRY</td>
</tr>
<tr>
<td>Kei AOKI</td>
<td>Chief</td>
<td>Hanshin Expressway Company Limited</td>
<td>JAPAN</td>
</tr>
<tr>
<td>Motohiko NISHIBAYASHI</td>
<td>Manager</td>
<td>Hanshin Expressway Company Limited</td>
<td>JAPAN</td>
</tr>
<tr>
<td>Hiroyoshi KOBAYASHI</td>
<td>General Manager</td>
<td>Hanshin Expressway Company Limited</td>
<td>JAPAN</td>
</tr>
</tbody>
</table>

| E-MAIL (for correspondence) | teruhisa-ishibashi@hanshin-exp.co.jp |

**KEYWORDS:**
Environmental Road Pricing
discounting the toll fare

**ABSTRACT:**
Road pricing scheme is widely applied in many urban areas such as Singapore and London as one of effective traffic management solutions to reduce the congestion and environmental degradation. Followed by the arbitrary conditions on the road pollution lawsuit against arterial surface road and toll urban expressway (Hanshin Expressway) between Osaka and Kobe, the road authorities are obliged to take any measures to reduce the cargo traffic from both routes. A unique road pricing scheme called "Environmental Road Pricing (ERP)" has been implemented since 2001. The ERP is to discount the toll of parallel Hanshin Expressway bay-side route, by which large vehicles are implicitly encouraged to shift to the cheaper route. The large vehicle traffic on the bay-side route gains the significant share, at present about 15 point increase from the commencement year. On the contrary, the traffic in target areas and the roadside air pollution level decrease steadily. The same ERP scheme is now being implemented in Tokyo area. The success of ERP is significant so that it can be recognized as a practical and effective traffic management approach by route choice of toll differentiation. It is applicable to other urban areas which suffer from the same problems.
1 INTRODUCTION

Road pricing scheme is widely applied in many urban areas such as Singapore (since 1975) and London (since 2003) as one of effective traffic management solutions to reduce the congestion and environmental degradation.

In Japan, Urban Expressway networks have been developed to handle the traffic in urban area. Followed by the arbitrary conditions on the road pollution lawsuit against arterial surface road and toll urban expressway (Hanshin Expressway) between Osaka and Kobe, the road authorities are obliged to take any measures to reduce the cargo traffic from both routes.

Therefore, a unique road pricing scheme called "Environmental Road Pricing (ERP)" has been implemented since 2001. The ERP is to discount the toll of parallel Hanshin Expressway bay-side route (Wangan Route), by which large vehicles are implicitly encouraged to shift to the cheaper route. Wangan Route is passing through the area with relatively less urbanized and less residential concentration compared with Kobe Route which runs parallel with it.

2 ATTEMPT START OF ERP

In Japan, expressway is classified into two major types, Intercity Highway and Urban Expressway. In urban areas, people and cars have increased with the growth of the economy, highway network to smoothly share the traffic in the city have been in place. In urban areas such as residential and office buildings had already been established, expressway networks have been developed by utilizing the public space of the existing road space or river space. However, large vehicle number has increased with the rapid growth of the economy, impact on the environment has become stricter. At that time, the development of the economy was rapidly, environmental regulations and measures for the office and automobile factories, etc. was not sufficient, roadside residents filed a lawsuit seeking improvement.

As a result, followed by the arbitrary conditions on the road pollution lawsuit against arterial surface road and toll urban expressway (Hanshin Expressway) between Osaka and Kobe, the road authorities are obliged to take any measures to reduce the cargo traffic from both routes.

Therefore, a unique road pricing scheme called "Environmental Road Pricing (ERP)" has been implemented since 2001. The ERP is to discount the toll of parallel Hanshin Expressway bay-side route (Wangan Route), by which large vehicles are implicitly encouraged to shift to the cheaper route.

3 OUTLINE OF ERP

In light of the above circumstances ERP has been implemented, the ERP is to discount the toll of parallel Hanshin Expressway bay-side route, by which large vehicles are implicitly encouraged to shift to the cheaper route.

The current discount is being processed in automatic toll collection system by the ETC, but at the time since the introduction of the ETC system had just begun, discount was also carried out by the discount ticket. Outline of Current ERP is shown in Figure 1. Large vehicle toll of Wangan Route in the target section is 30% discount is compared to Kobe Route. To promote the conversion of traffic, large vehicles passing through Osaka direction 10-15% discount applies.
4 TRAFFIC DIVERSION EFFECT OF ERP

Figure 2 shows the secular change of the large vehicle cross-section traffic volume. Due to the impact of the Lehman shock in 2009, the total traffic volume has been greatly reduced, later, traffic volume is recovering year by year. Since 2009, while the total traffic volume of Kobe line and National Highway Route 43 passing through the residential area is almost flat, the Wangan Route traffic is 27,100(Nov. 2013) units from 19,800 units (Mar. 2009) and to have increased 37%. In comparison with the ERP of the introduction initially, the total traffic volume of Kobe Route and Route 43 has declined 8,400 units in 2001 to 2013.

Figure 3 shows the transition of traffic share of large-sized vehicle traffic using Wangan Route, Kobe Route and National Highway Route 43 (which is the free surface road along the Kobe Route). This shows the clear effect of ERP implementation and appreciated by the stakeholders.

The large vehicle traffic on the bay-side route gains the significant share, at present about 15 point increase from the commencement year. On the contrary, the traffic in target areas and the roadside air pollution level decrease steadily.

Hanshin Expressway fee structure has changed in the price in accordance with the mileage from areas costs in January 2012. ERP is its importance, it will be carried out continuously with the contents of as before (no content that has been retracted than previously), has led to the current.

There are cases where road pricing is introduced as a means of eliminating traffic congestion. As a newly introduced technique, a technique of impact on the environment discounted toll relatively small line was able to convert a certain level of traffic. Such an approach, because the operating person does not give an economic burden, ERP is considered to be an effective method one.
Figure 2. Trends large vehicle traffic volume
6 CONCLUSIONS

In order to convert a large car traffic that passes through a residential area in less Gulf root of congestion, was introduced the technique ERP to discount the fee.

The total traffic volume of Kobe Route and Route 43 that passes through a residential area, has declined 8,400 units in 2001 to 2013. On the other hand, the Wangan Route that fee is discounted, the component ratio of 3 routes has increased 15 points. Technique to discount the toll of routes that are vacant, is effective as a method of road pricing.

The same ERP scheme is now being implemented in Tokyo area. The success of ERP is significant so that it can be recognized as a practical and effective traffic management approach by route choice of toll differentiation. It is applicable to other urban areas which suffer from the same problems.