IRF Global R2T Conference & Expo
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Bridge health index and asset management of bridge inventories

**Abstract:**
Bridge performance measures are an important component of successful bridge management system. They can be used as tools for bridge managers to assess structural vulnerability of bridges, manage and track the overall health of the bridge inventory. A bridge health or condition index is used as a performance measure by agencies interested in preserving the condition of bridge structures or prioritizing the maintenance or replacement projects within their bridge inventory.

The bridge condition or health index is a useful tool for assessing the structural or functional health of a bridge. The index is calculated based on the condition of the bridge’s structural elements and the service provided by the bridge. For purposes of bridge management, the most important use of the bridge health index (BHI) is to identify the bridges in the inventory that are most deteriorated and are mostly in need of repair.

The increased availability of element-level inspection information influenced the redevelopment of bridge health indices used in the US and around the globe. Currently, most Bridge Management Systems rely on element-level data for calculating BHIs.

The work presented herein uses the ratio based method approach developed by California Department of Transportation. This approach is used to generate a single number measure of the structural performance of a bridge or a network of bridges. The index assesses the current condition of a bridge by aggregating the current condition value of all the elements of the bridge and comparing it to the total value of the bridge elements when they were in their best possible state. The value of each element is proportional to the quantity of elements in the present condition and the economic cost of the element's failure.

Based on this approach a CapEx plan for an inventory of bridge and culvert structures has been developed to maintain the condition of structures based on industry practices and to implement into an Asset Management program. Assumptions made to develop the CapEx plan are based on detailed structure condition data made available, general observations and findings from site visits, and other information provided by the Client.
ABSTRACT:
The Mersey Gateway Project is one of the UK’s largest civil engineering projects and was opened Saturday 14th October 2017. It comprises a new six-lane cable-stayed toll bridge over the River Mersey and a 9.2km road network connecting the new bridge to the north west of England’s main motorway network. This paper provides details of the key operational management system which is also used to better understand and report on performance and finance – The Mersey Gateway Information System (IS).

The system was designed, developed and delivered by Nicander to deliver fully integrated asset, fault and operations management services, managing all key performance and financial information associated with the 6-lane bridge and its highway network. It is a web-based information management platform using the latest in virtualized and smart technologies to deliver a common operating picture and the effective management of information associated with:

- All highway and bridge assets
- All traffic information, control and journey time technology assets
- Activities of inspections, surveys and maintenance
- Winter service, weather and environmental requirements
- Emergency call-outs, enforcement and abnormal load routing

It provides a comprehensive information reporting service, including calculation of the financial payments (the Verification and Validation Service) based on all key performance indicators and the utilization of the bridge and vehicle speeds. This comprehensive data set is validated and processed through complex algorithms to determine all penalty information and complex KPI reporting procedures to calculate the monthly payment charge payable by the Authority to the operating company.
The Mersey Gateway Project is one of the UK’s largest civil engineering projects. It comprises a new six-lane cable-stayed toll bridge over the River Mersey and a 5.75 mile road network connecting the new bridge to the main motorway network in the north west of England. The bridge itself is 1.5 miles long with a river span of 0.6 miles. The main bridge deck is made from reinforced concrete and the spans are supported by steel cable stays attached to pylons rising up to between 80 and 125m above the river bed. The new bridge carries six lanes of traffic with a speed limit of 60mph.

The project is a major strategic new transport route linking the Liverpool city-region and the north-west to the rest of the country, and delivers major transport improvements including reductions in journey times, enhanced reliability and less congestion, alongside local social benefits such as improvements to public transport and walking and cycling facilities. Studies show that the economic benefits Mersey Gateway brings to the region are almost four times greater than its cost.

The local authority, Halton Borough Council, set up a new body the Mersey Gateway Crossings Board Ltd, to deliver the project. The Merseylink consortium was appointed as the project company by the council and is responsible for the design, build, finance and operation of the bridge over the next 30 years. The £1.86 billion lifetime cost includes the design, build, finance, operation and maintenance of the project through to 2044. The majority of the funding comes from the tolls paid by road users, but there is also a contribution from the UK Government.

The Merseylink consortium was appointed in March 2014 and is made up of experienced sponsors with a track record in delivering major infrastructure projects, alongside their partners, who are world leaders in their field. The sponsors are Macquarie Capital (Australia), FCC Construcción S.A. (Spain) and BBGI; the contractors are FCC Construcción, Kier Infrastructure & Overseas Limited (England) and Samsung C&T Corporation (Korea) and the toll operator is Emovis (France).

The bridge was opened just after midnight on 14th October 2017 as superstitions of Friday 13th got too much for some, with a fireworks fanfare as highlighted in Figure 1.
Figure 1 - Successful Opening 14 October 2017
2 Key Systems to Ensure Service Performance

From an Intelligent Transport Systems perspective, there are three key systems deployed on the project. These are fully integrated to ensure effective network management and service operations, and include:

- Journey Time Measurement System – to accurately measure journey times over four of the Project Roads at all times (24/7/365).
- Master Control System (MCS) – for the effective monitoring and management of incidents and traffic flow optimisation (CCTV, fixed and mobile variable message signs, automated incident detection, weather monitoring, lane control signs and traffic signals).
- Information System (IS) – asset and service operations management system, including the verification and management of all key performance indicators for financial reporting.

This paper focuses on the Mersey Gateway Information System (IS), designed, developed and delivered by Nicander.

3 MERSEY GATEWAY INFORMATION SYSTEM OVERVIEW

The Mersey Gateway Information System (IS) is a fully integrated asset, fault and service operations management system managing all information associated with the 6-lane bridge and its highway network.

It is a web-based information management platform using the latest in virtualized and smart technologies to deliver a common operating picture and the effective management of information associated with:

- All highway and bridge assets
- All traffic information, control and journey time technology assets
- Activities of inspections, surveys and maintenance
- Winter service, weather and environmental requirements
- Emergency call-outs, enforcement and abnormal load routing

It provides a comprehensive information reporting service, including calculation of the financial payments (the Verification and Validation Service) based on all key performance indicators, alongside utilization of the bridge and vehicle speeds. The system collates data from two ANPR systems and traffic counting equipment. This is validated and processed through a complex algorithm to determine all penalty information and across complex KPI reporting procedures to calculate the monthly payment charge payable by the Authority.

The system provides a GIS map for situational awareness. It interfaces with a common database for the collation of defined data and systems including ANPR, weather and traffic detection. The system monitors the status of all UTMC compliant devices including variable message signs, CCTV, incident detection, ANPR and traffic counters. The system also manages faults associated with the tolling system. The system interfaces to weather detection outstations to monitor visibility performance and then automatically reports against thresholds to adjust KPI reports appropriately.

The IS provides map views capable of plotting and showing the locations and current state of asset and equipment faults, planned maintenance and work orders. The map is made of multi-layered vector based and raster background maps thanks to OpenLayers and its capability of supporting map data from any source using OGC standards. It uses the geospatial capabilities of MS SQL Server database for rendering map objects symbols on the map layers. The map is fully scrollable and navigable offering pan and zoom features to navigate around the map in multiple web browser windows with the ability to easily switch between background maps (e.g. Open Street Maps, ArcGIS maps).

The system uses virtualized and Microsoft SQL Server and reporting products. A standard SQL database enables comprehensive flexibility to configure and access data to future proof the system’s implementation. SQL Server Log Shipping provides system resilience. The solution has defined database tables for National Street Gazetteer data.
The system architecture and core service functionality is summarized in Figure 2 below:

Core requirements for the system are scalability and accessibility. As the system is web based it is accessible from anywhere and by any authorized user with access to the internet, including running on tablets and smartphones. The system has been designed to easily cater for increases in the numbers and types of equipment throughout its life. The system servers are deployed in a virtualized infrastructure, conform to open industry standards and use facilities to provide high availability operation; it operates within a secured website using industry best practice, and utilizes the latest in mobile phone and tablet technologies for real-time connectivity and operation. Its features include fully customizable reporting and rules for sending alerts by email and SMS. Multiple web browsers are supported.

The client end of the system is made available over the public internet and therefore security against web based attacks is essential. Access rights defined within the system manage the specific roles and responsibilities of all staff; access privileges are assigned to users using groups, where a group will contain one or more access privileges. The user can belong to multiple groups and will inherit all privileges from all groups that they are associated with. Access to the system from both a desktop browser or from the mobile applications require authentication using a unique username and password; encryption is performed using an MD5 algorithm. All communication from the client to the server uses the MD5 encrypted form of the password and this communication will be further protected from packet snooping software by using SSL for all communication. A password reset option is provided which allows a user to specify their registered email address to receive a reset password.

The system provides a data configuration portal web user interface, which is accessible to a defined group of administrators, and is used to view and edit the configuration data used within the fault management system.

The technology platform comprises the Web, Application and Database servers alongside servers for Document Management & Version Control and File Management. Each server is deployed with a specific purpose to provide isolation at the operating system level allowing CPU, memory and disk capacity to be tailored for the server component. This architecture will easily allow the upgrade of servers at a later date should the need arise. The database server is an ACID compliant database, supporting full point-in-time database recovery and native backup/restore capabilities; it includes SQL Server management studio to provide features for monitoring, reporting and analysis of the server performance.
4 Evolving IS Requirements

At the time of initial tendering, critical operational performance parameters were mapped from the main contract documentation into very high level IS requirements. Quite simply, these were specified as bullet points highlighting areas of operation and maintenance services covering asset inventory, inspections, surveys and maintenance, winter services and environmental management, abnormal load and network management, public relations, journey time measurement, traffic control and toll collection.

It was the responsibility of the tenderer to use their expertise to formulate and price an overall solution to deliver these high level requirements. These were refined by the Consortium and further detailed as part of later phases of the tender process to provide additional clarity and understanding for all parties. Nicander were awarded a contract for the delivery of the Mersey Gateway Information System (IS) and were contracted by prime contractor Dynniq for this solution.

Figure 3 below shows the bridge spanning the river Mersey in North West England near Liverpool and Manchester.

5 THE INFORMATION INTEGRATION SOLUTION

The Information System collates key operational information through the integration of the Master Control System (MCS) covering:

- Traffic management technology equipment faults, control settings and road closures.
- Emergency call-out services, identifying key location information, all reporting and clearance timings, staff and officers involved in reporting and resolving the incident, road and weather conditions, and other relevant information.
- Lane closure information for carriageways, footways and cycle-ways required for emergency call-outs and roadworks.
- Traffic incidents on the project network, identifying date and time, specific location and lanes affected, weather conditions, vehicles involved and casualty details.

The Information System delivers asset management and maintenance functionality across the technology equipment, structures and infrastructure assets. It provides a web based set of forms for recording maintenance actions, such as inspections, survey, planned maintenance and reactive maintenance (Fault Work Orders) against the project locations and assets stored in the Information System. In addition it
provides a facility to define schedules and maintenance plans which can be created to facilitate the maintenance requirements defined in the various operational plans.

It incorporates a monitoring function to automatically raise a reactive maintenance work order (Fault Work Order) when a fault is received from the MCS system or a defect is raised during an inspection. In addition, notifications can be configured to immediately notify interested parties when work orders are raised in the system.

The system provides a facility to create assets, categorize the asset types and associate maintenance records to the asset inventory maintained within the Information System. The Maintenance Contractor can use the system to schedule or produce ad-hoc asset inspections, surveys and maintenance tasks.

For the structural elements the asset hierarchy can be used to create a hierarchy of the structural assets including major and minor structures, structure groups and key elements. Each of the structures can be associated with a location and the system provides a facility for providing the spatial/GPS location of the structural assets as well as its location on the highway.

The system categorizes the assets into:

- Paved areas
- Geotechnical assets
- Road restraint systems
- Traffic management technology
- Fences, walls and screens
- Verges
- Drainage
- Structures
- Lighting
- Roadside tolling infrastructure
- Road marking and studs

The system provides a facility for adding any category of asset or asset type and it is possible for each of the above categories to be further subcategorized. The ‘condition’ attribute associated with an asset is used to report any Service Failures deductions and can be used to provide the handback conditions of the assets. In addition, the system records the status of the current outstanding faults raised against the assets.

All appropriate asset management strategy, planning and supporting documents/drawings can be ‘attached’ to assets held within the system for improved operational understanding.

6 WORK ORDERS

Different categories of work orders such as planned maintenance, inspections, surveys, faults, abnormal loads as well as others can be raised in the system. These can be created manually or raised automatically as a result of an alert or fault received from the MCS system. Manually created work orders can be created as a one-off task or scheduled to occur in the future. Scheduled work orders can also be defined as a recurring task. The system allows filtering of the different types of work orders to be contained in the system. Some of the defined types of work orders include:

- Reactive maintenance work orders for the toll collection system and traffic control equipment
- Inspections and surveys work orders, including environmental inspections
- Planned maintenance work orders
- Public relation records
- Abnormal load routing work orders

The system allows custom inspections to be created together with an inspection schedule to define the checks to perform when inspecting assets such as street lighting, traffic signs and roadside tolling infrastructure. These inspections can be configured to raise maintenance actions to repair any defects recorded during inspections. The system will facilitate the entry of routine scheduled inspections as well as unplanned inspections. An unplanned inspection may be triggered as a result of information received or manually input from external systems such as the Bridge’s structural health monitoring system, from faults received from the MCS or as a result of failed inspection checks when engineers perform their inspection activities.
The target end date will be determined from configuration data for each inspection type and this is only editable if the operator is granted the appropriate access privileges. The system uses the target end date to determine if any inspections are overdue (not completed by the target end date). Overdue inspections are highlighted in the system and appropriate notifications sent to the mobile application on engineers’ phones.

The system allows each inspection type to be customized with a list of inspection checks and attributes for each defined type, including attributes to allow the entry of the asset condition. An inspection check can be configured to optionally raise actions to be performed when the inspection check fails. Details recorded during the inspections include date and time of inspection, required personnel, weather and asset condition information, a set of inspection checks and photographs. Inspection records are usually completed using Nicander’s mobile application.

The bridge Structural Health Monitoring System (SHMS) provides all of the required bridge monitoring and reporting facilities. The IS can be configured to raise an inspection work order as a result of any alerts observed on the SHMS operator interface, resulting in the system triggering an inspection task on the associated asset, defining the checks to perform and asset condition. The system will provide the facilities to create work orders such as inspections, surveys and planned maintenance on the network assets and associated information such as UKPMS inspection results, pictures or other supporting documents to the work orders and associated pavement assets.

The asset management aspect of the Information System provides the facility to enter scheduled cyclic maintenance as planned maintenance work orders on locations and assets. The system allows different planned maintenance types to be defined for the equipment and asset types.

The maintenance plan for each asset or location can be defined by scheduling a planned maintenance work order containing the planned start date, planned end date, the type of task required and, if necessary, the repeat period for the work order. If the work order has been provided with a repeat period, the system will automatically generate a new future planned maintenance work order on its completion. The control room operators and field engineers can record the progress of the planned maintenance work orders. The planned maintenance records include all relevant asset and location information, specific details of the tasks to be undertaken, supporting documents and photographic information, planned and actual times and dates, and traffic management requirements and health and safety information. The target start date and end date is recorded for each inspection and highlights when the inspections are overdue (not completed by the target end date).

The system provides a means of entering communications with external parties and recording any actions taken, which includes full details of all queries and complaints, status and reply details as well as any actions to be taken to meet the Service Requirements.

A reactive maintenance work order (e.g. for technology fault) can be created manually by a user or automatically by the system. Each received fault or alert type will be mapped to an internal code and this will be used to determine the severity and the target end date for the raised work order. The management of the fault is handled by the predefined workflow process which allows engineers to update the progress of the fault repair using either the web pages or more typically the mobile application. All actions performed in response to a work order are recorded and visible in the audit log and may also be visualized in a report. The engineer has the facility to provide additional details in the form of attached pictures or documents associate with the work order record.

7 MAINTAINING SCHEDULE

The system can be configured to hold a schedule of any routine inspections and planned maintenance to be performed on the assets and locations. This can be used to reflect the operational program and plans that are required to be produced by contractors. The system provides a target start date and end date for each inspection and the system will highlight when the inspections are overdue (not completed by the target end date). The program of environmental inspections on the relevant assets and locations stored in the Information System can be defined to reflect the Environmental Management Plan.
The system provides a number of mechanisms for notifying the interested parties. The primary mechanism for notifying the maintenance engineers, surveyors or inspectors is via mobile devices, which raise audible and visible alerts for any new reactive maintenance work orders or planned work orders that need to be actioned.

In addition the system can be configured to create notifications, using SMS and email, to provide information on the current state of work orders. The notifications are configured to send e-mail or SMS notifications to defined recipients based on certain rules. The actual messages, recipient details and the conditions for sending a message are configured in the Data Configuration Portal. When the defined rules result in more than one notification being suggested for recipients, the system will use the notification priority to determine which notification should be issued to each recipient. Any notification sent will be logged in the Information System database.

8 BRING YOUR OWN DEVICE

The Nicander mobile app (Android and iOS) allows engineers to receive alerts and new work orders, and view all outstanding work orders including those for automatic faults. The mobile application allows engineers to process all tasks through to clearance, providing details of any actions performed. In addition the mobile application allows engineers to complete inspection sheets which are submitted back to the main system for verification and approval. The mobile application is designed to work in areas of no data reception and will forward information to the server once a data connection is established.

9 THE PAYMENT MECHANISM

The purpose of the Payment Mechanism is to provide a monthly report for billing purposes. An annual fee payable to the Project Company (contractor) has been set which is payable each year over the 30 year lifetime of the contract. This is subject to annual indexation applied from the 1st April 2018 and each subsequent anniversary. The monthly payment is the appropriate proportion of the annual fee less any deductions that may apply. The deductions relate to the performance of the Project Company in respect of service, maintenance and keeping the Project Road usable.

The Information System extracts all of the relevant data required to produce the Monthly Unitary Payment and the Annual Unitary Charge. The IS comprises a complex model of all key performance indicators based on many parameters. The Deductions that may be applied include Monthly Journey Time Deductions, Operational Performance Adjustments, Lane Closure Charges and Retentions / Deductions in connection with commencement of operations and works completion.

One of the key components of the payment solution is based on the measurement of journey times across four key sections of the route.

The Information System stores the information received from two independent Automatic Number (or License) Plate Recognition (ANPR) based Journey Time Measurement Systems (JTMS) and an automatic traffic counter system. This information is used as a core component of the payment calculation and includes the journey time deduction thresholds and the portion of Unitary charge at risk. It details the total vehicle counts from two JTMSs and the traffic counter system at a time period for each of the collection points, with the sample size of the matched vehicles at the four route average journey time collection points, for each of the measured time periods. The system stores information covering the percentage of vehicles that have a travel time in excess of a configurable threshold and the length of any zero-flow period.

The validation and verification component performs cross checking of the vehicle registration marks (VRM) provided in both the JTMS1 and JTMS2 individual vehicle journey samples. Because it’s possible for a vehicle to perform the journey more than once within the 3 hour assessment period the matching is based on finding the matching journeys based on the vehicle registration mark (VRM) and end time of the journey. This is achieved by pairing the vehicle reference number from JTMS1 with those in JTMS2 with an end time within a defined tolerance. Any journeys without a matched pair are discarded. The time tolerance for finding the pairing is configurable and expected to be a number of minutes (far less than would be plausible for a vehicle to travel the route again i.e. 10 minutes). The output from this process is the average journey
times from the two JTMS systems, the paired sample size, average error value and standard deviation. Journey Pairings are only be performed on exact VRM matches.

The average journey time for JTMS1 and JTMS2 is calculated using the formula:

$$\frac{\sum_{i=1}^{n} \text{journey time}_i}{n}$$

Where n is the number of paired journey times. The average journey times for JTMS1 and JTMS2 are stored with the aggregated date produced for each assessment period on each assessment route.

An Average Measurement Error statistical value is calculated based on the difference between the average (Avg) of journey times from journey time 1 and journey time 2, i.e.:

$$\text{Avg. Measurement Error} = \text{Avg journey times JTMS1} - \text{Avg journey times JTMS2}$$

The Average Measurement Error is stored with the aggregated data stored for each assessment period on each assessment route. The variance of the average error is used when calculating the 95% confidence limit and is calculated using the formula:

$$S = \sqrt{\frac{\sum(x - \bar{x})^2}{n - 1}}$$

Where:

- $\bar{x}$ is the average measurement error
- $x$ is the difference between a paired travel time
- $n$ is the number of paired journeys.

The total vehicle count is taken from the traffic counting system. If this system is unavailable then JTMS 2 is used to supply the counts.

The journey time thresholds are re-baselined every six months to ensure payment reflects accurate and real journeys as utilization of the bridge changes over time.

10 CONCLUSIONS

The Mersey Gateway Bridge was successfully opened on time on 14 October 2017. The Information System was designed, developed and delivered by Nicander. It is currently delivering the following benefits to a wide range of stakeholders:

- By linking all asset information with key performance indicators and the financial payment mechanism, all stakeholders have the information they need to fully understand both infrastructure and operational performance.
- Integrating all key information into a single point provides a detailed insight into the management of all operational activities and enables management teams to quickly resolve issues ensuring contract performance requirements are met.
- Presenting the right information to the right people at the right time enables problems to be identified and resolved early and infrastructure to be maintained to the highest standard.
- Delivery of high quality and accessible performance data from which operators, managers and contractors can tailor their views of the system and generate reports which best suit their needs.
• Enabling the automation of effective identification, classification and prioritisation of faults/repairs, means that time consuming fault/repair recording tasks are automated and all activity is fully logged.
• Facilitating the identification of poorly performing technology and root cause analysis and enables maintainers to respond quickly to faults and avoid downtime to ITS infrastructure.
• Providing all parties with the tools to effectively plan all routine maintenance tasks and enables real time completion of inspection reports, improving the health of the infrastructure, staff utilisation and operational efficiency.
• Delivery of improved access to historic data to enable the identification and implementation of pre-emptive strategies to increase asset availability and value.

11 CITATIONS AND REFERENCES

1. http://www.merseygateway.co.uk/merseylink/
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ABSTRACT:
Condition ratings of bridge components in the FHWA’s Structure Inventory and Appraisal database are determined by bridge inspectors in the field. However, the determination of bridge condition ratings is generally subjective depending on individual inspectors’ knowledge and experience, as well as varying field conditions. Recently enacted government regulations for mandatory element level inspections (AASHTO, 2013) pose a need for effective record keeping and continued observations. This paper describes the results of on-site bridge deck scanning by digital imaging and infrared thermography technologies. In this research project, three infrared cameras with different specifications were compared for effectiveness in application to bridge deck scanning from a moving vehicle. The results obtained by three different infrared cameras were compared to show how each’s specifications have evident effects on the degree of accuracy in the detection of delaminations within concrete bridge decks. It can be concluded that for highway infrared inspection, the cooled infrared camera is far superior to other commonly implemented models, and a high exposure rate is critical in preventing issues like blurred imagery and subsequent false detections. The cooled model effectively doubled the window of time in which inspections can be carried out. By raising work efficiency and precision using cooled models, there are many benefits - reduction of field work hours leads to further reduction in field data collection costs. False detections and blurry images will be minimized, and with the support from the analyzing software the bridge inspection engineer gains much more intelligent data at a much faster rate.

Biography of the Author:
Masato Matsumoto is an expert on technical aspects of NDE bridge condition assessment, inspection, deterioration prediction, life-cycle expectancy and long-term highway asset management. Matsumoto is a globally recognized structural engineer, with a professional engineering (P.E.) license in the US. He is also a member of the IRF Asset Management Committee.
Bridge Deck Scanning by Infrared Thermography

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1 INTRODUCTION

Condition ratings of bridge components in the FHWA’s Structure Inventory and Appraisal database are determined by bridge inspectors in the field. However, the determination of bridge condition ratings is generally subjective depending on individual inspectors’ knowledge and experience, as well as varying field conditions. Recently enacted government regulations for mandatory element level inspections (AASHTO, 2013) pose a need for effective record keeping and continued observations. This paper describes the innovative nondestructive bridge deck inspection technology using high-definition infrared and visual imaging. This combination of instruments benefits from rapid and large-scale data acquisition capabilities. Depending on the target structure, different systems have been developed to combat on-site accessibility issues and long man-hours in the field associated with conventional sounding methods. FLIR hardware and proprietary software was integrated into a mobile system called the Deck Top Scanning System (DTSS). This system identifies thermal and visual variances indicative of deficient areas within concrete. By introducing advanced functions in automatic detection, image stitching, and geo-referencing, the system makes for practical and adaptable solutions to manage inspection data (Watase et al. 2015).

The daily fluctuation of radiation into and out of the Earth’s atmosphere has allowed structural inspectors to study thermal patterns within concrete, brick, and asphalt surfaces. The technology makes full use of this natural occurrence by conducting inspections during day and night. Infrared thermography (IRT) captures irregularities in these fluctuations generated by flaws such as cracking, spalling, or delamination of internal structural layers. Alongside a synchronized visual record, it can prove a very powerful tool for structural analysis. We strongly encourage the use of some form of visual technology alongside IRT, due to discoloration, oils, or other anomalous occurrences that can create false positives within IRT data. Given a visual technology with high enough resolution, it is possible and practical to conduct the visual inspection of cracks and spalling at the very same time.

Through its implementation in Japan over the course of two decades, the technology is opening new possibilities in a field with much untapped potential. In this paper, findings and lessons learned from our past experience in Virginia and Pennsylvania are described as examples of mobile NDE (DTSS) in action.

2 IMPLEMENTED TECHNOLOGY

The Deck Top Scanning System (DTSS) features an IRT camera and two line-scanning cameras (LSC). This equipment is mounted onto an inspection vehicle, from which the system can capture visual data during the daytime and IRT data during proper testing periods in either the morning or evening. As a result, the DTSS rules out the necessity of lane closures and direct visual inspection. This promotes safety and efficiency in the field by steeply reducing the time which workers and engineers are exposed to highway dangers on-site (Hiasa, Et al. 2016).

These instruments are accompanied by both IRT image processing and crack detection software, called Ir-BAS and JeEditor respectively. The software packages perform data collection, image correction (an orthographic representation of an image taken at an angle), image stitching (the forming of one composite image with many smaller images), GPS coordination assignment, automatic damage detection, and other analysis functions. The DTSS has classically been applied to concrete structures such as bridge decks and parking garages.
With the accumulation of data from repeat tests, controlled and uncontrolled, as well as first-hand experience on site, accurate monitoring of the progression of cracking, spalling, and delamination is possible. The technology is accurate enough to calculate the changes in length, width, and/or area of deficiencies over practical time periods (six months to a year, for example). Also, the nature of the resulting data contributes to a visual record of entire structural surfaces, which can be pragmatically saved and referred to in the future. With a comprehensive visual record of target structures, more predictive power in life cycle models and cost models will drive wiser future planning.

2.1 Infrared Thermography Technology, Automatic Detection, and Processing Software (Ir-BAS)

The recommended camera model for IRT scanning is the FLIR A6701sc model, capable of taking thermographs at high speeds due to its fast exposure rate and integration time (similar to the shutter time of a visual camera). It can also detect very finite differences in temperature (down to 0.1°C), making it possible to conduct inspections at night, when temperature differentials between sound and unsound regions tend to be quite slim.

The core of our infrared analysis technology is a software called Ir-BAS (which also includes the stitching software IrLay). It implements a comprehensive database comprised of past bridge inspection sites. The software translates infrared data into algorithmically sorted images, displaying points of potential delaminated areas in real time (as displayed in Figure 2 below). Conventionally, IRT images must be reviewed by an experienced technician during long hours of post-processing, but our program’s auto-detection function offers a quicker alternative. It emphasizes thermal anomalies and pronounces large temperature variations within naturally occurring temperature gradients. This pronunciation may identify deficiencies which would be otherwise overlooked while reviewing large amounts of data.

Refer to Figure 2 and 3 below. Raw thermographic imagery can only display temperature readings as they are, incorporating a wide range of temperatures over a certain area. Doing so will unfortunately mask thermal anomalies, making them difficult to spot. IrBAS software detects gradients and effectively filters them, uncovering what lies in their midst. The software retains the original temperature data but makes it much easier on the viewer to pinpoint areas of interest.
temperature is determined in very small groups of pixels and used as a basis for the adjusted display. This process is portrayed graphically in Figure 3 below.

![Figure 3. Effects of thermal gradient extraction (IRT image processing)](image)

It is through this operation which unsound areas are discriminated from sound areas: by overhauling the IRT data itself and analyzing temperature differential in groups of pixels. An advanced numerical model, which can be adjusted within program settings, automatically picks out areas in which temperature differentials cross a certain threshold (Akashi et al. 2010). For example, in the left-hand graph in Figure 3 above, the temperature of the sample anomaly is around 18.4°C, whereas the temperature of the proximate sound area is around 17.9°C. By taking the average temperature around this anomaly and calculating the trend of the background temperature gradient, a new field (or scale) of temperature is produced, essentially neutralizing the gradient. The previous 0.4-0.5°C differential is therefore magnified in the right-hand graph. Depending on the IRT camera used, the minimum detectable differential may change, but the process within the software remains the same.

Ir-BAS is also responsible for image correction and stitching. Refer to Figure 4 below for a systematic diagram of workflow. GPS data taken on site is used to properly orient and place images, and the resulting string of images can either be viewed on a geolocated plot (which accurately maps the pathway and direction of the pass), or vertically (aligned in a straight line). With the stitched imagery, it is possible to export into a desired image file type and be compared side-by-side with the visual line camera scan.

![Figure 4. IRT image stitching process](image)

2.2 Visual Proofing and Crack Detection with High-definition Line-cameras

The line-scanning camera (LSC) equipment is used to produce a high-quality visual diagram of bridge decks, and in turn a visual proof for the IRT data. Frequently, things such as discoloration, oil spills, or other markings can result in false detections when reviewing the IRT thermographs, so it is necessary to compare both visual and IRT data side-by-side (or by overlaying one on top of the other). Cracks down to a 0.3mm width can be recognized on the line scan, and generally these detections are measured and drawn manually onto the images by a trained engineer.

Each individual camera scans at a 13 ft width (around 18ft in total, with 8ft overlapping), in a constant stream. The camera requires motion, ideally a steady speed to operate, but the frequency and spacing of recorded lines are adjusted in real-time by a high-performance speedometer. See more details about the LSC in the table below.

The line data is processed and analyzed with an image manipulation and crack editing software called JeEditor. Based on intervals determined by speedometer and GPS, the software arranges the visual line data (rows of ~10-100 x 4010 mm² pictures) and provides an interface which the user can overlay scaled gridlines and deficiencies. The deficiencies can be classified based on type, area, width, etc. Various light-correction functions allow the user to...
enhance the image and make hairline cracks easier to detect. Areas/lengths drawn in the interface are quantified, and can be output to excel for report composition.

![Figure 5. Image editing software interface](image)

An LSC system is the recommended alternative to high resolution area-scanning photography or video, and exceeds ASTM D 4788-03 standards. In our experience, the DTSS using LSC and IRT in combination has provided higher resolution image results (resolution: 1mm²/pixel) in assessing and documenting defects, and does not result in blurring commonly observed in area-scanning cameras.

3 FIELD VALIDATION TEST

3.1 Project location

The project diagnosed the condition of a bridge deck on the Pencoyd Viaduct (I-76) approaching Philadelphia along the Schuylkill River. The project scope included deck top scanning for identification and quantification of delaminated or spalled areas. The data acquired was validated by sounding test during a lane closure which took place nearly two weeks after the initial scanning. The bridge in question is shown in Figure 6 below.

![Figure 6. Pencoyd Viaduct in Philadelphia, PA](image)

The field data collection for IR images was performed between 8 pm to 10 pm on April 18th, 2017 (Figure 7). Digital imagery was collected during daytime (between noon and 1 pm) on April 19th. The reason behind taking both a visual and an infrared scan is the necessity of visual proofing, or checking for false positives. While interpreting IR scanning results, it is important to differentiate and subsequently exclude false positives on the deck surface caused by things such as spilled paint, roadway striping, or other unusual objects. Without making this comparison, it is possible to misinterpret a discoloration as a delamination, because they tend to exhibit similar temperature differentials.
3.2 Field data collection

The total scanning hours in the field for digital imagery was approximately 50 minutes for both West bound and East bound bridges, including shoulders and two lanes. The nighttime IR scanning was conducted in the same manner in about 50 minutes. To ensure a successful scan with IR technology, it is critical to select the best time window for the field data collection, in respect to temperature conditions. The best and most stable time window generally occurs from the start of an intensive thermal flow from the concrete to the air at sunset, and lasts until the temperature differential between the concrete and air stabilize. Differentials and thermal anomalies can be detected at this time using an infrared camera with a high thermal sensitivity and fast exposure time. In general, this thermal flow is created by both ambient temperature and radiative cooling at night. Figures 8 and 9 below illustrate the behavior of temperature and radiation within concrete structures over the course of a day.

![Figure 7: Digital image scanning](image1)

![Figure 8. Ambient and concrete temperature variation a day](image2)

![Figure 9. Thermal flow in concrete structures during day and night](image3)
A temperature record taken near the site is displayed in Figure 10. The temperature of ambient air (light blue) and a sample test piece concrete (red and green for unsound and sound concrete, respectively) was recorded at a parking lot near the inspection site. During the time of the IR scan (8 pm – 10 pm), approximately one-degree Celsius gap between sound and unsound concrete occurred.

Figure 10: Temperature graph recorded on April 18 and April 19. IR scanning period is shown in blue box.

3.3 Findings from the high-speed scan

Favorable weather conditions and a smooth scan resulted in a successful analysis of the bridge. All data collected—raw infrared, processed infrared, and visual—were compared closely side-by-side to eliminate false positives. The AASHTO Manual for Bridge Element Inspection, First Edition (AASHTO, 2013) provides a guideline for condition state definition of bridge elements based on the severity of delamination, spalling, exposed rebar, and cracking conditions. For the East bound bridge, the element condition state (CS) for reinforced concrete decks (Element #12) was determined based on the mobile scanning results. Broadly, this bridge exhibited only slight signs of defects compared to the overall bridge roadway deck area - less than 2% of deck area with some delamination. The distribution and location of the AASHTO-categorized deficiencies for each bridge was also prepared in the form of a map, where the deficiencies were highlighted and super-imposed onto an image of the bridge decks. Percentage of deck areas for each condition state was also summarized for each span of the bridge and graphically displayed as shown in Figure 11. This information can be used by bridge owners as a general assessment, and can help to prioritize future repair/rehabilitation programs by identifying deficient areas in their early stages.
Figure 11. Condition state distribution for each span of the bridge

The result from the high-speed scanning was displayed on the deficiency map for details (see Figures 12 and 13 for examples). The tables in the deficiency map show the percentage of deck areas for each condition state. Two weeks later, the right lane and the shoulder of the East bound bridge were closed for visual inspection and sounding tests for validation purposes (Figure 14). Figure 15 is the validation results depicted on the deficiency map for the East bound bridge. The areas marked in orange color are detected as delamination by the sounding test. The areas in yellow color are detected as potentially delaminated by the high-speed infrared scan. We added GREEN dashed boxes if these orange and yellow areas were consistent (that means correct detection by infrared). However, some of the yellow areas were proved to be "false positive", and we added RED dashed boxes to these false positives. Generally speaking, the defective areas were successfully detected by the infrared with only two exceptions in Span 2 and Span 10. In these areas, a signature of delamination was found by the IR scan, but we interpreted these areas as false positive on the basis that exposed aggregate and a continuous discoloration on the middle of the deck surface camouflaged this area and it was challenging to distinguish this area from the other sound areas which showed a similar temperature distribution. Most of the false positives were also caused by the same reason.

Figure 12: Deficiency map for Span 7 of the East bound bridge
Figure 13: Deficiency map for Span 8 of the East bound bridge

Figure 14: Sounding probe used for the validation test
3.4 Conclusions and recommendations

The result from the high-speed scanning for the concrete deck for the Pencoyd Viaduct was displayed on the deficiency maps, and will be further examined as the analysis of the viaduct continues in the coming year. The results were validated by the sounding test applied during lane closure, and provided an excellent opportunity to cross check our results, and heighten our knowledge of signatures of delamination. The sounding test proved that most of the defective areas were successfully detected by the infrared thermography technology. However, the exposed aggregate due to the abrasion of concrete made it challenging to distinguish the delaminated and sound area from the infrared results, causing some false detections. From the validation results, it can be concluded that high-speed infrared and digital image scanning can provide valuable information of current deck conditions with reasonable accuracy. The information provided for identification and mapping of deficiencies will serve as an invaluable reference for the purposes of future evaluation and comparison. The bridge owner can use the comprehensive deficiency map for monitoring distressed deck areas and planning future repairs, as well as summarizing AASHTO element level inspections in square footage or percentage of deck area. The deficiency map and element condition summary generated by the deck scanning technology can serve as a solution for more efficient, objective, and safer bridge inspections in comparison to traditional approaches, especially for relatively large-scale bridges with heavy traffic volume. To assure the reliability of IR data, it is recommended to select the best field data collection time periods and to use an IR camera equipped with a cooled detector. Using enhanced analytical tools, the engineer can pinpoint locations of deficiency, allowing more efficient inspection protocols, improved operational safety, all while driving lower life cycle costs for owners. Other non-destructive evaluation technologies and any hand-held scans can also be applied for more detailed condition assessment for the selected areas denoted by high-speed scanning technology. Combining the different type of NDTs depending on their data collection speed, accuracy and cost will provide better solutions for bridge owners to efficiently and properly monitor the bridge conditions, supporting data-driven (objective) decision making on bridge maintenance and management.

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The Development and Evolution of the Steel Bridge Fracture Control Plan

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ABSTRACT

In 1967, the Silver Bridge in Point Pleasant, WV over the Ohio River collapsed, resulting in 46 fatalities. The catastrophic collapse brought bridge safety to the forefront of national attention. Numerous changes resulted from the collapse including the introduction of the National Bridge Inspection Standards (NBIS). Ultimately, the collapse would become the defining event of the original steel bridge fracture control plan (FCP). Over the following 50 plus years, research and experience would continue to shape the FCP into its current state. The present study is a synthesis of the development and evolution of the steel bridge FCP. Beginning with the Silver Bridge collapse, the review will examine events shaping the steel bridge fracture requirements, specifically in regard to material toughness, design review, and in-service and shop inspection. Recently, there has been a renewed interest in steel bridge fracture research. A discussion of current and ongoing studies will conclude the synthesis. The current work has begun to re-examine the traditional steel bridge fracture approach in an attempt to develop new standards aimed at implementing modern materials, design, fabrication, and inspection. In addition, the recent research has employed technological advances including finite element analysis to more accurately assess the potential for fracture leading to collapse. Combining the historical events and present work into a single comprehensive review will provide engineers a necessary understanding of steel bridge fracture and the intention of the current specifications.

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HISTORY OF THE 1967 SILVER BRIDGE COLLAPSE

The Point Pleasant Bridge over the Ohio River, also known as the Silver Bridge because of the coated aluminum paint, was constructed in 1928 (Witcher 2017). Connecting Point Pleasant, West Virginia and Kanauga, Ohio, the overall bridge length was 533 meters. At the time of construction, eyebar suspension bridges had been designed for nearly 100 years. Many of these past bridges were built with redundant bar links. Redundant eyebar members employ thinner bars of modest material strength, intertwined to complement each other. However, the eyebars in the Silver Bridge were non-redundant. The links were composed of two high strength steel bars with twice the tensile strength of common mild steel (Talbot 2011).

The AASHTO LRFD Bridge Design Specifications, defines redundancy as “the quality of a bridge that enables it to perform its design function in a damaged state” and a redundant member as “a member whose failure does not cause failure of the bridge. (Hartmann 2015)” AASHTO LRFD also notes redundancy can be supported by load-path redundancy, structural redundancy, and internal redundancy. A member is considered load-path redundant if an alternative and sufficient load-path is determined to exist. In other words, if a member were to fail, the adjacent or parallel redundant member would be sufficient enough to carry the redistributed load. A member is considered structurally redundant if its continuity or support conditions are such that failure of the member merely changes the system behavior but does not result in the collapse of the superstructure. For example, if a two-span continuous bridge girder were to fail in the negative-moment region, it would not be critical if the positive-moment region were strong enough to carry the load as a simply supported girder (Hartmann 2015). A member is considered internally redundant if a sufficient cross section exists within the member itself to carry the load in the event of failure of one of the elements (Hartmann 2015).

On December 15, 1967, the Silver Bridge collapsed resulting in 46 fatalities. All three sections of the bridge span fell within seconds after loud cracking sounds were heard. The cause of collapse was determined to be a brittle fracture of one of the non-redundant eyebars supporting the main span. A similar sketch of the failing C13N eyebar chain joint can be seen below in Figure 1. The eyebars were constructed of heat-treated rolled carbon 1060 steel bars with forged heads, designed to fail in the shank at ultimate load (Rossow, 2015). At the time of the collapse, temperatures were around -1°C. There was no required material specification at the time of the bridge design. Test results demonstrated the steel used would not pass current design codes. Corrosion fatigue was thought to initiate cracks at the inside of the pin hole of eyebar C13N (NTSB 1970). In addition, in 1941 the original timber bridge deck was replaced by a steel grid with approximately eight centimeters of concrete, not accounted for in the dead load calculations of the original design. Ultimately, the fracture toughness was not sufficient and a small flaw led to a catastrophic collapse.

Figure 1: Eyebar Chain Joint, connection to truss by hanger strap plates (NTSB 1970).
INTRODUCTION OF THE NATIONAL BRIDGE INSPECTION STANDARDS (NBIS)

The collapse of Point Pleasant Bridge influenced engineers, designers, and legislation to investigate greater protocols and to analyze the condition state of all current bridges in operation. Collaboratively, the Federal Highway Administration (FHWA) and the American Iron and Steel Institute (AISI) sponsored research to address issues of non-redundant steel bridge members for brittle fracture (Hartmann 2015). Around the time of the bridge collapse across the Ohio River, welded bridge construction standards and material standards in the era were very lightly regulated (Altstadt 2014). There were no national material requirements for base metal, and in-service field inspections were rarely performed.

Congress passed the Federal-Aid Highway Act of 1968 instructing the United States Department of Transportation (U.S. DOT) to create the National Bridge Inspection Standards (NBIS) (FHWA 2008). The standards were officially adopted in 1971 and would begin to answer questions of how, when, and who were to conduct bridge inspections. Visual assessments were to be performed every two years unless otherwise warranted by conditions. Not until 2001 would this code requirement be updated to an exact 24-month calendar time cycle for bridge inspections. For Non-National Bridge Inspection structures in 1971, inspection was to occur at least every 60 months. Inspectors were to be well-qualified with at least five years of bridge inspection experience. Additionally, inspectors were to complete a two-week comprehensive training course for fracture critical members (Thurber 2015).

DEVELOPMENT OF INITIAL STEEL BRIDGE FRACTURE REQUIREMENTS

The Point Pleasant Bridge collapse also provided the impetus for additional fracture research (Barsom 1970). The standard test for fracture performance evaluation in the steel bridge industry is the Charpy V-Notch (CVN) impact test. This test provides a measure of the impact energy required to break a material under impact loading at a predetermined temperature. CVN impact testing is an indirect measure of material toughness. Correlation equations exist to tie CVN energy to fracture toughness. In addition, CVN testing demonstrates if a material is expected to be ductile or brittle at a given temperature by the temperature-transition curve (Independence 2017).

A Charpy impact test uses a mechanical arm freely dropped from an initial starting position in a pendulum motion to impact a test specimen. Test specimens are small bars containing a specified v-notch with a small radius at the base of the notch (ASTM 2005). The notch serves as the point of fracture initiation. The striking arm will impact the specimen and continue to swing through to the other side of the machine. The height of the follow-through swing indicates the amount of energy absorbed. Figure 2 presents a schematic of a Charpy impact test and the standard v-notch in a Charpy test specimen.
Fracture toughness research throughout the 1970s included AISI Project 168, a steel industry investigation for the toughness criteria of bridge steels. The testing was performed at the U.S. Steel Research Laboratory (Barsom 1975). This study considered typical bridge loading rates, temperatures, plate thickness, and the CVN-to-KIC correlation, where KIC is the plane strain mode I fracture toughness as defined by ASTM E399. The testing included four full-scale beam tests and numerous CVN impact tests, leading to the first AASHTO fracture-toughness specification for bridge steels in 1974. The initial CVN requirement was 15 ft-lbs (Barsom 1975). In addition, three temperature zones were established, defined by their respective Lowest Anticipated Service Temperature (LAST). The zones are still in use; however, they are now referred to as the Minimum Service Temperature (MST) by AASHTO.

While the 1974 specification identified a major policy shift towards brittle fracture elements, there was not a clear conscious of how to achieve adequate fracture resistance. From 1975 to 1979, debates argued a few major issues were not addressed in the work leading to the 1974 specification. Of the concerns not addressed, many researchers were skeptical about pop-in cracks as a failure mode. Pop-in cracking is the dynamic formation and propagation of a crack that emanates from a local brittle zone (Hartbower 1979). Two groups investigated the claim. Group 1 hypothesized pop-in cracking was a major risk and CVN requirements should guarantee the steel can prevent small cracks from spreading at local brittle zones. Their methodology for proving this assumption was to establish specific CVN energy values and ductility requirements at the respective LAST. Group 2 hypothesized pop-in cracking should not be a concern. Their approach was to specify CVN values such that macroscopic fatigue cracks at typical bridge strain rates would not lead to immediate plane strain brittle fracture (Barsom 1975). The CVN requirements would be tested above the LAST region because of the strain rate difference between the dynamic CVN test and quasi-static bridge loading rate.
Years of research demonstrated strengths and weakness of each hypothesis. Group 1 claimed the static loads on bridge structures were equal to the bridge loading rate. However, Group 2 argued pop-in cracks could not cause such a large failure in design fatigue cracks. Fracture would need to initiate from a combination of low toughness steel, large flaws in design, or fatigue cracks emanating from poorly designed connection details, such as welding errors. (Fisher 2001). Testing performed by Group 2 suggested pop-in fractures were infrequent based on field and laboratory testing. Yet, Group 1 indicated bridge structural steel have high toughness resulting in the low incidence of fatigue cracks.

Group 2 tested 24 beams at low temperatures with large cracks at typical bridge strain rates (Roberts 1977). The beams were loaded beyond the code-specified design load and no failure occurred. It was reasoned that the beams did not fail in design because of the strength of the structural steel material. Rather, it was due in part to a cyclic crack growth. Cyclic cracks grow at an exponential rate that increases with crack size; therefore, the greatest crack growth rate is immediately before fracture. Group 2 would come to win the 1970s debate over pop-in cracking. Group 1 agreed additional fatigue cycles after a fracture would not greatly increase the total life of the structure. However, if improvements were made to solve future fatigue crack growth life effects, inspectors would be able to better identify in-service cracking. For all fracture critical members (FCM), samples from each plate would be tested to ensure adequacy. Non-FCMs would still require testing, specifically at regions of high stress as identified by during design.

In 1978, AASHTO would release the Fracture Control Plan (FCP) that became a very comprehensive guide aimed at reducing the likelihood of brittle fracture (AASHTO 1978). The FCP covered design review, material toughness, fabrication, welder certification, and weld inspector qualifications. The 1980s continued steel bridge fracture toughness research. With the LAST temperature zones established in the 1970s, the desired minimum level of fatigue cracking to be judged was not significant enough to understand the effects of CVN strength on FCM. Therefore, CVN test temperatures were reduced by 11°C, and the established requirement for CVN sampling was increased. The requirement was added to remove specimens from each end of the as-rolled plate (Frank 1993). Literature indicates the reasoning on the exact temperature reduction was more of an engineering judgement than based on experimental data.

1983 MIANUS RIVER BRIDGE COLLAPSE

The Mianus River Bridge was a pin and hanger bridge constructed in Greenwich, Connecticut in 1958. On June 28, 1983, a 30 meter suspended span of the eastbound traffic lanes collapsed and fell 21 meters into the lower river as shown in Figure 3. Two semitrailers and two automobiles plunged into the water resulting in three fatalities and three serious injuries (Burnett 1984). From the investigation of the National Transportation Safety Board (NTSB), before the collapse of the suspended span, the inner hanger in the southeast corner of the span came off of the inner end of the lower pin. The weight then became distributed to the outer hanger which was not designed to sustain the entire load of the assembly. Over time, the added weight initiated a fatigue crack. The outer hanger shifted farther outward on the pin and as soon as it reached the fatigue crack, the shoulder of the pin fractured off and the entire assembly failed causing the suspended span to collapse.
The NTSB determined the probable cause of the collapse of the Mianus River Bridge span was the undetected lateral displacement of the hangers in the southeast corner of the span by corrosion-induced forces due to deficiencies in the bridge safety inspection and bridge maintenance program in the State of Connecticut (Burnett-1 1984). The investigation revealed the drains on the bridge were difficult to keep open because the hydraulic slopes of the piping were too shallow and changes in direction were too abrupt. Drain placement was inaccessible and difficult to repair without proper mechanical equipment. However, the largest drainage issue occurred when the roadway was resurfaced in June of 1973 and the drains were covered with steel plates (Burnett-1 1984). The steel plates were used to protect the grates from the asphalt resurfacing, however, the plates were never removed after the maintenance was completed. As a result, the bridge was only being drained through the expansion joints (Burnett-1 1984). Although inspections reported signs of water draining along the pin and hanger assembly of the bridge, none of the accounts were corrected. The final conclusion for the Mianus River Bridge failure was that the pin and hanger assembly failed because of corrosion.

THE DEFEAT AND REBUILD OF THE NBIS TODAY

Since the creation and adoption of the NBIS by the FHWA in 1971, many questions were raised on how effective these standards were to protect bridges from failure. The original NBIS grew from the Point Pleasant Bridge collapse and was based on the “Manual for Maintenance Inspections of Bridges,” published in 1970 by AASHTO (Burnett-2 1984). The standards require that:

1. All States have a bridge inspection organization;
2. Inspectors meet minimum qualifications;
3. Each structure be rated as to its safe load-carrying capacity;
4. Inspection records and bridge inventories be prepared and maintained in accordance with the standards; and
5. Every bridge in a public road be inspected at regular intervals (not to exceed 2 years).

The Mianus River Bridge collapse led to these standards being re-examined. The bridge was in the state of Connecticut jurisdiction and followed guidelines from ConnDOT. Unfortunately, some of the guidelines contradicted those in the NBIS. Specifically, the inspectors were to be well-qualified, whereas documentation shows some of the inspectors were classified as junior inspectors. Supervisors were to review the junior inspector reports; nevertheless, both parties continued to ignore the signs of corrosion problems. Prior to the Mianus collapse, bridges across the country were to be inspected every 2 years. This was not very specific and made the requirement quite lenient. An inspector would be able to inspect a bridge in January 1972 and would not need to check it again until December 1974 because it was still dated “2 years” apart from 1972-1974. In reality, this was closer to 3 years between inspections.
In September of 1986, the NBIS released guidelines for “Inspection of Fracture Critical Bridge Members”. Following, in September 1988, the NBIS was modified, based on suggestions made in the “1987 Surface Transportation and Uniform Relocation Assistance Act,” to require states to identify bridges with fracture critical details and establish special inspection procedures (MnDOT N.D.). In December 1988, the FHWA revised the “Coding Guide” for inspectors to provide them with additional direction in performing more uniform inspections and to document accurate findings. In part because of the 1983 Mianus River Bridge collapse, this uniform inspection required all bridge inspectors to use a hands-on inspection of fracture critical members. This requirement significantly increased life-cycle costs relative to non-fracture critical members (Connor 2005).

In 2002, the Bridge Safety Inspection Training program was revised with a new manual named the Bridge Inspector’s Reference Manual (BIRM) and was later updated in 2011 to inspect fracture critical members and culverts. States had become concerned with the meaning of bridges to be inspected every two years and recommended flexibility should be given to adjust to unexpected weather events, or to permanently move a bridge or group of bridges to a more logical inspection period (FHWA 2004). In addition, Arkansas asked for a 45-day grace period after the two years if weather caused an issue for inspection. In December of 2004, the FHWA declared the hands-on inspection frequency for fracture critical members should “not exceed” 24 months. The FHWA went into further clarity by stating, “We recognize that severe weather, concern for bridge inspector safety, concern for inspection quality, the need to optimize scheduling with other bridges, or other unique situations may be cause to adjust the scheduled inspection date. The adjusted date would not extend more than 30 days beyond the scheduled inspection date, and subsequent inspections should adhere to the previously established interval (FHWA 2004).”

2000 HOAN BRIDGE FAILURE

On December 13, 2000, Hoan Bridge located over the Milwaukee River in Milwaukee, Wisconsin, was examined after fractures were discovered in all three girders of one of the southern approach spans. The interior and east exterior girders were fractured full-depth as shown in Figure 4 (Cooper 2001). Explosive demolition removed the span and portions were sent to the FHWA Turner-Fairbank Highway Research Center (TFHRC) and Lehigh University for evaluation. The investigation conducted structural analysis, local stress analysis, fracture mechanics analysis, and live load testing (Cooper 2001). During a review of the bridge history, inspectors reported what were thought to be fatigue cracks in 1995 in other locations of the bridge, which were addressed at the time with a repair. However, the failure investigation revealed no fatigue cracking prior to fracture initiation. The failure mode was identified as a brittle cleavage fracture (Cooper 2001). Further investigation revealed a narrow gap between the gusset plate and the transverse connection/stiffener plate created a local tri-axial constraint condition and increased the stiffness in the web gap region at the fracture initiation site. The constraint prevented yielding and redistribution of the local stress concentrations ultimately resulting in the fracture (Cooper 2001). New AASHTO design details resulted from the failure to avoid constraint-induced fracture. In addition, retrofit procedures were been developed and implemented for the existing inventory.

Figure 4. Framing and elevations plan for the northern end span identifying the cracked girder locations (Fisher 2001)
MODERN FRACTURE TOUGHNESS TESTING

Since the early 1960s, many years of research have relied upon Eq. (1) to analyze through-thickness cracked laboratory specimens using linear elastic fracture mechanics (Anderson 2005). The main goal of the equation, as well as the research conducted from the 1970s to the 1990s, was to investigate if plane strain fracture would occur. Although the equation sees limited use today, it was created at time before modern nonlinear analysis became available. Bridge engineers and scientists of the 1970’s felt the criteria for separating faulty brittle steel plates from acceptable plates was sufficient from this equation. However, more recent research has demonstrated sudden, violent cleavage fracture can occur regardless of whether Eq. (1) is satisfied or not. Eq. (1) cannot identify when a plain strain condition is or is not present around a cracked plate specimen (Anderson 2005).

\[ B \geq 2.5 \times \left( \frac{K_{IC}}{\sigma_{YS}} \right)^2 \]  

\( B \) = plate thickness

\( \sigma_{YS} \) = yield strength

\( K_{IC} \) = plane strain fracture toughness

More vigorous methods for fracture mechanics are being used in other industry standards, such as BS7910:2013 (BSI 2013), API 579 (API 2016), and R6 (British Standards 2013). For example, the master curve is an exponential function which represents the lower-shelf and lower transition region fracture toughness for all ferritic steels. The master curve only shifted the reference temperature \( T_o \), which is the temperature at 100 MPa√m (Wallin 2007). Correlations exist in the previously mentioned specifications to correlate CVN impact energy to \( T_o \). The master curve can be paired with a thickness correction for different plate sizes as well as statistical tolerance bounds. Such methodologies have the potential to enhance the current fracture prevention strategies in the steel bridge industry.

CURRENT/FUTURE ERA OF STEEL FRACTURE SAFETY

Since the early 1970s, work continues to be made by the steel bridge industry to advance design policies thereby promoting safety from unseen hazards and fatal accidents. Modern bridge designs show much improvement. Good engineering must be used in design, detailing, material selection, and in-service inspection of those members currently classified as fracture critical. Since 2010, there has been a renewed interest in fracture critical research. The 24-month hands-on inspection interval is costly in both time and resources. Further, the safety of the inspectors and motoring public are important considerations relative to the value of the detailed inspections. Historically, fracture critical bridges built since the inception of the fracture control plan have performed remarkably well. Three primary studies have resulted in the areas of system-level redundancy, internal redundancy, and high-toughness steel.

Computational power has grown exponentially since the creation of the original FCP. Non-linear finite element analysis is now able to accurately capture complex behavior of non-redundant structures. As such, the FHWA has defined a member classification known as a system redundant member (SRM). SRMs are members shown through analysis to have system-level redundancy. As such, a fracture of an SRM would not result in catastrophic collapse of the bridge. However, SRMs are still designed and fabricated per the FCP. The new member definition required research to establish analysis procedures, failure criteria, and load and resistance factors. The work has recently completed and published data should soon be available for public use.

A second project recently completed with coming changes to fracture critical specifications involve internal redundant members (IRM). Historical evidence has demonstrated built-up members in the field have survived fracture of a single component without the fracture jumping to additional components. Research quantified such behavior through large-scale testing. The testing also included establishing the fatigue resistance in the faulted states. The research has confirmed built-up members are capable of surviving a dynamic fracture event. When sufficient fatigue life remains, the detailed inspection for built-up components can be extended well-beyond the current 24-month interval.
Aside from analysis, there has been many other advances in the 40 years since the original fracture control plan was released in design, materials, fabrication, and inspection. The original FCP considered each of these under a single document; however, each component has since been separated into standalone specifications. While, the current provisions have proven to be successful in preventing fracture, an integrated approach considering design, material, fabrication, and in-service inspection is being developed. The approach relies upon modern advances, specifically high-toughness steel. Through the research, an integrated FCP will allow owners to rationally set inspection intervals to ensure steel bridge safety, while promoting an efficient use of owner resources.

CONCLUSION

The tragic collapse of the Silver Bridge was the beginning of great change in the bridge industry. Major outcomes from the event included the development of the NBIS and original 1978 FCP. It also began decades of research into steel bridge fracture provisions. Currently, non-redundant steel bridges are inspected at an arm’s length every 24 months to ensure failures are avoided. However, advances in design, materials, fabrication, and inspection now allow for a more advanced approach to steel bridge fracture control. Such advances are further complimented by non-linear finite element analysis and decades of an outstanding service record. The advances since 1978 when the FCP was initiated now allow fractures to be treated like any other reliability based limit state. Recent research into SRMs and IRMs have made great strides toward improved fracture provisions. Ongoing research into an integrated FCP with rational inspection intervals has the potential to further improve steel bridge safety. This historical review has shown that the steel bridge fracture provisions will continue to adapt with technological advances and improved understanding. The goal for the future is to guarantee safety for all bridge designers, bridge inspectors, and the users of all bridges throughout the world, while promoting an efficient use of owner resources.
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ABSTRACT:
The concept of ‘soft load testing’ (SLT) was introduced in the project SAMARIS (Sustainable and Advanced MAterials for Road InfraStructure), through development and implementation of the new generation of Bridge Weigh-In-Motion (B-WIM) systems. Its main purpose is to emulate traditional diagnostic of proof load tests: to measure the true response of the structure under loading and to update accordingly the analytical model of the bridge accordingly. Using B-WIM systems means no need for pre-weighed vehicles to load the bridge, as during a traditional diagnostic load test. Furthermore, there is no interference for the users during measurements as the bridge is left opened for the traffic. And, due to the lower load levels, there is no risk of damaging the structure, one of the main concerns with other load tests. SLT primarily acquires the realistic influence lines, which, particularly on older shorter bridges, differ considerably from the theoretical ones. The most common reasons are restricted movements of the supports and insufficient knowledge about the design or construction details at the time of assessment. Further improvements to the structural model can be achieved by measuring the girder distribution factor (GDF), which aims to provide distributions of traffic load for a sufficiently high number of loading events that these can be statistically evaluated. Finally, from measuring the responses of all vehicles the real dynamic amplification factors (DAF) can be assessed.
INTRODUCTION

The Bridge WIM (B-WIM) method was introduced almost 40 years ago (Moses, 1979) but has never penetrated the market as could have been expected based on its advantages. It became popular only in Australia, where it evolved into a system that replaced bridges with culverts (Peters, 1984; Peters, 1986). Based on Moses’ theory two prototype B-WIM systems were developed independently in the early 1990’s in Slovenia and in Ireland. Some work has also taken place in other countries, particularly in Canada and Japan (OBrien et al., 2008), and lately again in the USA. B-WIM was extensively studied in the late 1990’s in two European research projects: the COST 323 action “Weigh-in-Motion of Road Vehicles” (2002) and especially in the Work Package 1.2 of the European Commission 4th Framework Programme project “WAVE – Weighing of Axles and Vehicles for Europe” (WAVE, 2001). This research emerged in a commercially available B-WIM system, called SiWIM®, that is constantly being improved (Žnidarič et al., 2010; Žnidarič et al., 2017).

Weigh-in-motion systems have been traditionally used to collect freight traffic data to support transportation planning and decision making activities. As high axle loads are responsible for road and bridge damage, the aim of any WIM system is to obtain accurate axle load and gross weight information. Despite the dynamic interaction between the vehicles and the pavement, which affects accuracy of WIM results, weighing in motion is recognized as the only method which can measure axle loads of the entire population of vehicles on a road section, including the overloaded ones which successfully avoid other modes of weighing. It is therefore the most efficient way to collect unbiased data on heavy freight vehicles. There are two major groups of WIM systems on the market, the ones that weigh with sensors built into the pavement and the bridge WIM systems.

B-WIM systems are applied on existing bridges or culverts which are transformed into undetectable weighing scales (WAVE, 2001). For this purpose the structures are instrumented with strain measuring gauges and axle detectors. Strains are collected on the main longitudinal members of the bridge to provide response records of the structure under the moving vehicle load. Until today, the traditional axle detectors on the pavement have been mostly replaced with FAD, the free-of-axle detector instrumentation, which uses additional sensors under the bridge to capture axle information.

Measurements taken at typically 512 readings per second during the entire vehicle crossing provide redundant information (more data points than needed), which facilitates evaluation of the axle loads, in particular when uneven road surface induces vehicle vibration. This is an advantage over the pavement WIM systems where, with exception of the multi-sensor installations, an axle measurement lasts only a few milliseconds. Bridge WIM is particularly appropriate for:

- Short-term (up to several weeks) measurements, as it can be easily installed and detached from the structure. Unlike the situation with pavement WIM systems, accuracy of the results of portable installations is not different than that of permanent installations.
- Measurements on sites, where cutting into the pavement is not allowed or is not feasible due to the heavy traffic.
- Bridge assessments, as, in parallel with weighing, the measured responses allow to evaluate some true key bridge performance indicators: the influence lines, the degree of dynamic amplification of the bridge under traffic loading and its distribution across the structure.

Data collected with B-WIM measurements are used for a number of applications, including traffic studies, pavement and bridge design and assessments, detection of overloaded vehicles, etc. The commercial B-WIM system, called SiWIM®, was developed in Slovenia as an outcome of research performed in the COST Action 323 (2002) and 4th Framework Programme project WAVE (2001), both financially supported by the European Commission.
BRIDGE WIM AND BRIDGE ASSESSMENT

Safety assessment of existing bridges is driven by many challenges. This is particularly valid for older structures where drawings and relevant construction information are often limited or even do not exist. However, as bridges have hidden reserves in resistance that are typically not accounted for during their design stage, they are most likely still safe, despite being deteriorated and accepting heavier traffic loading. As a result, the objective of bridge safety assessment is to verify that a structure has adequate capacity to safely resist specific loading levels/effects. Or, in other words, to identify structures which probability of failure exceeds those prescribed in the codes (BA 16/97, 2001; EN 1991-2:2004, 2003; CAN/CSA-S6-00, 2005; AASHTO LRFD, 2007) or those accepted by the structure owner/manager. The aim of efficient assessment is to gradually employ more and more detailed information, accompanied by material testing and measurements of loads and of behavior of the bridge under the loading.

With the aim to make structural analysis quicker, more reliable and cost effective, projects SAMARIS (Sustainable and Advanced Materials for Road Infrastructure) from the European Commission 5th Framework Programme (SAMARIS D30, 2006) and ARCHES (Assessment and Rehabilitation of Central European highway Structures) from its 6th Framework Programme (2009) proposed two new approaches: soft load testing (SLT) and optimized evaluation of the dynamic amplification factor (DAF). Both are based on B-WIM measurements and enhance the results of bridge assessment, with the ultimate goal to rationalize spending of resources for road infrastructure maintenance.

SITE SPECIFIC TRAFFIC LOAD MODELLING

Reliable WIM data source used for traffic load modelling on bridges requires at least 100,000 heavy goods vehicle records (Žnidarič, 2017). Furthermore, the individual vehicle records must be recorded with 1/100 of a second precision or better, which is not provided by all WIM systems. SiWIM gives time stamps of vehicle records at 1/512 of a second.

CALCULATION OF STATIC LOAD EFFECTS

Static load effects, \( Q_s \), are calculated for all relevant loading events on the bridge, by combining the axle loads of the measured vehicles with the bending and shear influence lines (for details, see the section on soft load testing):

\[
Q_s = \sum_{i=1}^{N} AL_i I(x_i)
\]

where \( AL_i \) is the weight of the axle \( i \), \( N \) stands for the number of vehicle axles and \( I(x_i) \) is the value of the influence line due to axle \( i \) at location \( x \). This approach was proposed by a number of authors (Moses, 1979; Karoumi et al., 2006; Žnidarič et al., 2010; Ieng, 2015). In theory, the shape of the influence lines depends on the bridge span and its boundary conditions (simply supported, continuous bridge etc.).

SIMULATION OF EXTREME LOAD EFFECTS

Results used in structural assessment must be presented in a form of maximum expected moments and shear forces at critical sections of the bridge. Most often, a number of statistical distributions, like Gumbel, Fréchet or Weibull, and even Normal, are used to fit the tail of load effects distributions and then to extrapolate to the ultimate expected values (OBrien et al., 2015). Alternatively, very detailed simulations can be applied to forecast the extreme load effects (Enright & OBrien, 2013).

On the other hand, the SiWIM software contains a bridge assessment module, which is built around convolution, a statistical method that assumes that the maximum load effect on a bridge is achieved with one vehicle in each of the two lanes of traffic (Moses & Verma, 1987; Žnidarič & Lavrič, 2010; Mandić et al., 2017; Žnidarič, 2017). This limits its application to bridges with influence lines shorter than approximately 30 to 40 meters. This, however, includes well over 90% of all bridges (Žnidarič et al., 2011).

In convolution, the static load effects given are presented as histograms for each lane, creating two independent probability mass functions, denoted as \( f_i \) and \( f_j \). Both functions are convoluted to obtain the probability mass function \( f \) that represents all possible joint load effects generated by pairs of vehicles from both lanes:
In the next step, the cumulative probability density function $F_z(z)$ is calculated. Finally, to determine the extreme load effect $Q_{\text{max}}$ in a selected time period $z$, typically 25 years for limited and 75 years for normal lifetime, the extreme value theory is applied (Ang & Tang, 1975):

$$Q_{\text{max}}(z) = (F_z(z))^{N_T}$$

(3)

where $N_T$ is the number of critical meeting events in the selected period. $N_T$ is calculated as the daily number of multiple-presence events (with one vehicle in each of the two adjacent lanes), times the number of days per year taken in consideration, typically 250 working days, times the selected time period (in years) for which we are calculating the expected maximum load effects.

**SOFT LOAD TESTING**

The concept of ‘soft load testing’ (SLT) was introduced in the project SAMARIS (2006), through development and implementation of the new generation of bridge B-WIM systems. Its main purpose is to emulate the traditional diagnostic load tests: to measure the true response of the structure under loading and to update accordingly the analytical model of the bridge. Using a B-WIM system means no need for pre-weighed vehicles to load the bridge that are required to perform a traditional diagnostic load test. Furthermore, during measurements the bridge is left opened for the traffic. Finally, due to the lower load levels, there is no risk of damaging the structure, one of the main concerns with other load tests.

SLT primarily acquires the realistic influence lines, which, particularly on older shorter bridges, differ considerably from the theoretical ones. The most common reasons are restricted rotations of the supports and insufficient knowledge about the design or construction details at the time of assessment. Further improvements to the structural model are achieved by measuring the girder distribution factors (GDF), to provide a true distribution of the traffic loads across the bridge. These are evaluated statistically from a sufficiently high number of maximum bridge responses to crossing vehicles.

In addition to its enormous potentials, the SAMARIS project identified the following limitation of SLT:

- It is not intended to predict the ultimate state behavior of a bridge but to optimize its structural model used for safety assessment.
- The validity of bridge assessment is generally short-term (from a single specific event to a few years or a couple of decades), especially if it is based on performance measurements which evolution is difficult to forecast. As SLT measures response of the structure that depends on changing conditions (degree of constraints in the supports, dynamic excitation due pavement roughness, etc.), its validity should also be limited to up to 20 years.
- For the same reason its results should only be evaluated by experienced bridge engineers who can predict bridge behavior under heavier loading and under potentially changing conditions.
- If the expected traffic loading is likely to exceed the one measured during the SLT by more than 50%, it is recommended to extend the measurements or to perform a diagnostic load test.
- It has only been tested and used on bridges shorter than 50 m.

**CASE STUDY**

In order to demonstrate how the results generated by a B-WIM system can be used for optimized bridge assessment, a bridge in the Mississippi, USA was instrumented with the SiWIM® system. The purpose of measurements was to provide necessary information to the bridge manager to:

- optimize the structural model of the bridge, to do more accurate structural assessment,
- reduce or even abandon bridge posting that may have been imposed due to its poor condition; thus, direct and indirect costs of bridge users can be considerably reduced,
- facilitate planning and monitoring of exceptional heavy transports.

The case study bridge is a typical US interstate bridge, composed of six simply supported steel girders and a composite reinforced concrete deck over them. It is 27 m long and 8.5 m wide and has transverse steel ties placed at the mid-span and at the quarters of the span. It carries two lanes of traffic. Being in good condition, it was not an ideal case to demonstrate benefits of SLT. These would be more evident on older bridges, which do not have proper expansion joints and bearings and which true behavior can differentiate considerably from the design assumptions.
The superstructure was instrumented with 10 strain transducers and without axle detectors on the pavement. Six sensors were attached to the bottom flange of each girder, close to the mid-span, to capture ‘global’ strains under vehicle loading, as required by the B-WIM algorithm to evaluate axle loads (Moses, 1979; Žnidarič et al., 2017). At the same time, these sensors provided information for the key performance indicators: influence lines and girder distribution factor. Four additional strain sensors were glued to the slab between the girders to capture sharp peaks from the individual moving axles (Figure 2, right). The pavement on both approaches to bridge was very rough, with a number of large potholes and loose asphalt aggregate (Figure 3).

**EXPERIMENTAL INFLUENCE LINES**

In bridge engineering, an influence line defines the variation of a load effect (bending moment or shear forces), at a specific point of the structure, that is caused by a crossing unit load. As such, it is one of the key components of B-WIM algorithms (Moses, 1979; ARCHES D08, 2009; Žnidarič et al., 2015). It has been shown that weighing with B-WIM (COST 323, 2002; OBrien et al., 2008) and methods for assessment of bridge performance (SAMARIS D30, 2006; Žnidarič et al., 2015; Mandić et al., 2017) are more accurate and effective if the influence lines are not calculated from theoretical principles but are based on measurements. Figure 4 compares the theoretical influence line of the case study bridge with the measured influence line that was generated by the SiWIM® system (Žnidarič et al., 2017). The third one results from the finite-element model in which springs were added, to match rotational constraints in the supports, and its cap was rounded, as a result of the superstructure’s depth.
Figure 4 confirms that even on a bridge of almost ideal simply supported design (with steel hinges at both supports), some constraints in the supports exist, which prevent the bridge from performing according to theory. These differences, and the consequent reduction of bending moments, can exceed 100% on older bridges where bearings and expansion joints do not exist, are damaged or otherwise do not work as anticipated. In the case bridge the maximum value of IL decreased for around 25%. It should be noted that such behavior not only lowers the bending moments but also slightly increases the shear forces. Once the calibrated structural model has been calibrated, it is straightforward to develop influence lines for any load effect and for any locations of interest on the bridge.

SITE-SPECIFIC TRAFFIC LOAD MODELLING

Figure 5 presents the histograms of bending moments for the driving and fast lanes which have been calculated from the measured axle loads and spacings and the modelled influence line, using equation 1. Similar histograms were calculated for the shear forces. On continuous bridges they would be also evaluated for the moments above the intermediate supports. It should be noted that the measurements made on the case study bridge were relatively short and thus the results are only indicative.

The extrapolated extreme bending moments over various time frames, as derived with equation 3, are shown in Figure 6, for the measured and the theoretical simply supported influence lines.

As summarized in Table 1, applying the measured influence line reduced the bending moments by 23 to 24%, compared to the values derived from the theoretical influence lines. Despite the difference not being as high as it could have been experienced on some other types of bridges, this bonus should not be neglected when assessing structural safety of the bridge. At the same time, the shear forces increased by 1% after applying the modelled influence line.
Table 1: Comparison of bending moments calculated with measured and theoretical influence lines, in kNm

<table>
<thead>
<tr>
<th>Period</th>
<th>1 event</th>
<th>1 year</th>
<th>10 yrs</th>
<th>25 yrs</th>
<th>75 yrs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean moments measured IL (kNm)</td>
<td>1820.7</td>
<td>2397.9</td>
<td>2920.3</td>
<td>3094.8</td>
<td>3285.3</td>
</tr>
<tr>
<td>Mean moments theoretical IL (kNm)</td>
<td>2385.2</td>
<td>3141.0</td>
<td>3797.5</td>
<td>4015.4</td>
<td>4256.0</td>
</tr>
<tr>
<td>Ratio between the measured and theoretical moments</td>
<td>76.3%</td>
<td>76.3%</td>
<td>76.9%</td>
<td>77.1%</td>
<td>77.2%</td>
</tr>
</tbody>
</table>

MEASURED GIRDER DISTRIBUTION FACTORS
The SiWIM® system also evaluates statistically the lateral distribution of loads at the points of measurement, which are typically around the mid-span for the bending moments and close to the supports for the shear forces and moments above the supports. Typically, the maximum values of a few hundreds of structural responses caused by single heavy vehicles crossing the bridge are averaged and the coefficients of variations of these results are evaluated to derive the experimental girder distribution factors (GDF). The average GDF values of the four instrumented girders of the case study bridge are given in Figure 8.

![Figure 8: Girder distribution factors for traffic in driving and overtaking lanes](image)

CONCLUSIONS
Bridge WIM has become increasingly important among other WIM technologies, not only for traffic classification, but also for the assessment of in-service bridge performance. Their main advantages are full portability, high accuracy due to the large weighing platform and the Free-of-Axle Detector (FAD) technology, which does not require putting any measuring devices on or even in the pavement.

Load testing provides key inputs to calibrate structural models. As performing a traditional diagnostic load test is expensive and requires closing the bridge during measurements, the soft load testing was proposed as a cost-efficient alternative of the traditional procedure. With the same objective, to collect information about the true bridge response under traffic loading to optimize the model, it uses a B-WIM system to collect the necessary information from normal traffic rather than from pre-weighed vehicles. The key performance indicators that are evaluated from measurements are the influence lines and the girder distribution factors (GDF). Both can considerably reduce the load effects that are used in structural assessment. In addition, being measured and not assumed, they also allow a reduction in the safety levels (selected partial safety factors or risks of failure) used in the analyses.

The SLT approach has been successfully used on tens of bridges in several countries, to show that traffic loading was less than prescribed in the codes and that bridge behavior was more favorable than it could have been concluded from the design and construction details. SLT is not performed on all bridges, but only on those where conventional analysis has failed to provide satisfactory results. As an example, structural safety of over 90% of evaluated bridges in Slovenia was proven sufficiently high to avoid the most severe rehabilitation actions, like strengthening or replacement of the bridge, which saved the owner tens of millions of dollars (Žnidarič et al., 2015).

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Beyond Developing RAMS – Contending Issues In Its Application

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ABSTRACT:
Beyond developing RAMS – contending issues in its application

Road network is an asset of national and strategic importance and underpins the overall socio-economic development of any nation. In most economies, road authorities and agencies must maintain, operate, improve, replace and preserve this huge asset base amidst scarce financial and human resources needed to achieve these objectives. More so road users are increasingly demanding for improved levels of service in terms of safety, reliability, environmental impact and comfort. Governments are placing greater pressures on road administrations to improve the efficiency of, and accountability for, the management of the road network. Balancing competing resource demands to deliver safe and reliable journeys on the Nigerian road network is a challenging task. Nine years ago, the Federal Roads Maintenance Agency (FERMA), a statutory Road Agency of the Federal Government of Nigeria, commenced a programme of work designed to implement an integrated road asset management system (RAMS). The objective was to deliver an efficient and cost-effective maintenance and management of over 36,000 km federal road network and 1,900 bridges, thus enabling asset managers meet their overall business and operational objectives. The architecture of the RAMS was designed with a view to enhance the capabilities of the asset managers by providing a source of readily accessible, relevant and valid information on the road as well as improved support for decision-making by providing analytical tools. The paper discusses contending issues in the deployment of RAMS to rationalize decision making in planning, programming, procurement and execution of works, and in the allocation of resources to ensure best use of public funds in preserving the road network at an acceptable level of serviceability.
INTRODUCTION

National economies rely on roads and highways particularly in meeting the mobility and economic needs of local communities, regions, and the nation. The road network is an asset of national and strategic importance and is often the single most valuable publicly owned national asset. Developing nation like Nigeria, the most populous country in Africa with a population of 195 million people on a land area of 911,000 km², has made an enormous investment in road infrastructure in the second half of the 20th century. The Nigerian road network, shown in Figure 1, has a total length of about 200,000 km of Federal, State and Local and rural roads and over 1,900 bridges which provide service to road users. It is largest road network in West Africa and the second largest south of the Sahara with a road density is 0.2 km per square km (0.06 km per square km (West and Central African). The road networks are planned along the north to south and east to west grids, to link major seaports in the south, airports, busy cities and major economic centres.

It is estimated that 80% of the Nigeria’s transport and 70% of socio-economic activities are over the road network. This investment has helped drive Nigeria’s economic growth – but maintaining this huge infrastructure is now proving to be an enormous financial strain. Despite the network being a valuable national asset, there has been a huge maintenance backlogs due to very limited and unpredictable funding - thus the roads have deteriorated considerably, and any maintenance funds available are used for reactive ‘fire-fighting’ activities, which are insufficient to maintain the quality of the road asset, preserve its value or minimize asset depreciation. This situation has arisen because the development of the road network in Nigeria has been characterized by a cycle of build, leave and then rebuild; there has not been a strong culture of preventive maintenance. Without properly planned maintenance programmes, the high value of any road network is quickly eroded and consequently becomes a cost to the economy.

The National Integrated Infrastructure Masterplan (NIIMP) estimates that about USD 350 Billion will need to be invested in the construction, rehabilitation and maintenance of the road network over the next 30 years to provide Nigeria with the road network its economy requires. If such a level of investment takes place, it is essential that Nigeria has the people, equipment, and decision-making tools necessary to direct the investment and maintain the value of the road asset.
2 BACKGROUND

Since the first ever Nigerian motorable road linking Ibadan and Oyo was constructed in 1906, the road network has increasingly become more sophisticated with features that include pavement, bridges, drainage structures, electrical appliances and appurtenances, road furniture, traffic control signals, safety barriers, earthworks etc. These assets are increasingly stressed from over-use, under-funding, and aging. A more holistic management approach is required to address the growing and increasingly complex challenges that lies ahead. If the maintenance of the network is inadequately funded or poorly planned there can be serious economic and social consequences for society in terms of access to essential services, road user delay, increased vehicle operating costs (including fuel consumption), and reductions in air quality. Studies have shown that maintenance interventions become more substantial and far more expensive as the condition of a road deteriorates. Unplanned, reactive maintenance, often the situation in Nigeria, is far more expensive than a planned programme of appropriate and timely maintenance. For example, where defects are neglected an entire road section may fail completely, requiring full reconstruction at three times or more the cost, on average, of maintenance costs (Burningham & Stankevich 2005). Similarly, the South African National Road Agency Ltd. (SANRAL, 2004) estimated that repair costs rise to six times maintenance costs after three years of neglect and to 18 times after five years of neglect. Good asset management can provide substantial cost savings for a road authority. Road authorities around the world have developed computerized Road Asset Management Systems (RAMS) to support their decision making, enabling them to develop rational, evidence-based maintenance plans that optimize the value obtained from maintenance investment. Most major economies have a RAMS.

Put simply, the objective of the asset management process is to provide the required level of service (in terms of the availability and condition of roads) in the most cost-effective manner. The level of service is usually specified by the owner of the asset (often the road authority). For example, it might specify that the extent of potholing or road roughness will not exceed a certain threshold or that a specific route will always have a minimum number of carriageways available to traffic. The objective is to determine the maintenance programme that meets the level of service at minimum cost. Effective road asset management is about doing the right amount of maintenance (i.e. the minimum necessary to maintain the required level of service) at the right time. For a large and varied network this is a very complex task, thus the utilization of computerized RAMS to support the decision making, enabling them to develop rational, evidence-based maintenance plans that optimize the value obtained from maintenance investment. Maintenance plans can be developed for individual sections of road, or a prioritized programme of activities can be developed for local, regional, or national networks.

There are three key elements required for an effective RAMS process:

1. **Asset information**
   Data collection surveys to provide regular, up-to-date information on the type and condition of the assets to be managed, and the levels of traffic loading they are subjected to (and which causes them to deteriorate)

2. **Decision support tools**
   Software and procedures that analyze the asset information, condition and traffic data to develop optimum maintenance plans

3. **Users**
   RAMS requires a body of staff who are experienced in:
   - Data collection survey techniques
   - Use and maintenance of survey equipment
   - Data processing and analysis,
   - Operation of the decision support software, and
   - Interpretation and implementation of its output

All three elements are necessary for deployment of the RAMS to provide optimum maintenance decisions, based on reliable information, which result in significant cost savings and improved performance of the network. This, in turn, provides confidence that money is being well spent, and attracts further
investment from both domestic and international sources, leading to continued improvement in network performance, which in turn underpins economic growth.

3 FERMA RAMS PROGRAMME

The Federal Roads Maintenance Agency (FERMA) is a statutory Road Agency of the Federal Government of Nigeria. The Agency in 2009 commenced a programme of work designed to implement an integrated road asset management system. The programme was supported by DFID-funded Nigeria Infrastructure Facility (NIAF) Programme and cuts across all three of the key elements of a RAMS – data collection, decision support tools and capacity building. The objective was to deliver an efficient and cost-effective maintenance and management of over 36,000 km federal road network and 1,900 bridges. This was expected to enable FERMA asset managers meet their overall business and operational objectives. A substantial programme of work completed in the last nine years has provided the building blocks required for a sustainable RAMS solution in Nigeria, particularly in relation to the technical aspects. Implementation of the RAMS has involved many tasks, some of which are still on-going, but the work completed to date has included the following activities:

Client needs analysis. Workshops and meetings to determine client needs in relation to the management of the road network. This covered the range of surveys and equipment needed to provide the necessary road network data for a RAMS, the functionality and format required for the decision support tools, and the structure of the organization required to host, manage and implement a RAMS.

Data collection. Key data required for effective asset management were collected using the following equipment:

- 2 No. Falling Weight Deflectometers (FWDs) to measure pavement deflection (from which pavement strength can be calculated)
- 1 No. Bump Integrator to measure road roughness
- 2 No. Image Collector vehicles to assess road condition visually, and record inventory items (e.g. surface type, road width, street furniture etc.)
- 3 No. Weigh pads to measure vehicle axle loads
- Bridge inspection equipment

FERMA Survey teams, formed and trained under the programme, carried out the initial network-wide surveys under supervision to obtain the inventory and condition data required for the RAMS. This included

- Pavement strength surveys: 22,000 km,
- Road roughness surveys: 19,000 km
- Image Collector surveys: 10,000 km
- Traffic counts (7 day, 24hr): 266
- Axle load surveys: 12
- Bridge inspections: 105

Subsequent surveys on the Federal road networks have covered:

- Pavement strength surveys: 8,800 km,
- Road roughness surveys: 8,700 km
- Image Collector surveys: 8,700 km
- Traffic counts (7 day, 24hr): 177
- Bridge inspections: 420

Development of software specification. Based on the needs analysis, a comprehensive specification was produced for RAMS software that met the needs of the Nigerian road sector. TRL’s iROADS system met most of these requirements and was selected as the Decision Support Tool. The software licence was provided free in perpetuity to ensure sustained application and avoid downtime associated with non-payment of licence renewal fee. This implies that the Agency would never be required to pay licence fees for use of iROADS, thus will help to build a sustainable RAMS process for the future. To meet the needs of stakeholders, additional areas of functionality were incorporated into an enhanced iROADS system. This
included the ability to import/ export data to and from HDM-4 (the internationally recognized economic assessment tool for road management) and a bridge management module.

**Configuration of iROADS.** The iROADS software was configured and customized to meet the requirements of the client in terms of the way data is stored, presented, analyzed and reported. A major task involved the production of a digital geo-referenced base map of the country’s Federal road network. This was a substantial piece of work involving referencing several data sources (including our own survey GIS traces) to compile one. This was completed for most of the network (approximately 32,000 km of road) and is shown in iROADS over the background mapping (Figure 2). Work continues to expand the geo-referenced Federal road network towards the expected 36,000 km - 37,000 km.

![Figure 2. Digital geo-referenced road network base map](image)

The inventory and condition data collected to date have been attached to the base map for analytical purposes. An example is given in the screenshot (Figure 3) shown below which shows pavement strength for sections of the Federal road south of Kano.

![Figure 3. Pavement strength for sections of the Federal road south of Kano](image)

The system allows data to be colour coded according to value – in this case green for high strength, amber for moderate and red for low. The surface condition on this road is very poor, and on that evidence alone the recommendation might be to carry out expensive rehabilitation works or even reconstruction. However, the strength data shows that the underlying structure of the pavement is relatively sound (most
sections are green or amber) so a much cheaper surface treatment may be all that is required. The FWD Survey team reported that there are significant sections of the network where the surface is poor, but the pavement structure is strong. The pavement strength data in iROADS has enabled FERMA to recommend far cheaper surface treatments at sites that were scheduled for more costly rehabilitation works.

4 RAMS DEPLOYMENT – CONTENDING ISSUES

Nigeria is a very challenging environment in which to introduce and sustain the concept of road asset management. Despite the substantial progress made to date on the development and deployment of RAMS for cost-effective management of Nigeria’s federal roads and maintenance planning, there still exist some challenges to contend with regarding its embedment to ensure good and sustainable asset management practice. The challenges are interlinked but are broadly categorized as institutional and technical challenges.

Institutional Challenges

Until recently, there has been no overriding and coherent road sector policy in place and guidance on the road prioritization process based on objective criteria. Consequently, the road sector operators and organizations lack guidance on how to deliver their mandates and prioritize activities in developing and maintaining the network. It is essential to continuously have a clear line of sight and alignment of road development projects and road maintenance programmes to road network development plans and road sector policy objectives and performance targets to avoid or minimize politically motivated decision making. Regarding institutionalization of RAMS, the recent passage of the road sector reform bill represents the first clear and deliberate step towards utilizing RAMS in the decision-making process and managing the road network in Nigeria. The bill, amongst others, require the new Federal Road Authority (FRA) to develop and utilize an appropriate RAMS for the day to day and strategic planning and evidence-based reporting processes. It further requires the Authority to prepare, maintain and monitor a 5-year rolling road maintenance and development plan in line with national development objectives, prioritization criteria and performance expectations.

Funding is another serious challenge. In Nigeria, the funds allocated to the road sector are heavily influenced by political considerations and are not evidence based – i.e. they bear little relation to the size and condition of the existing network. Funding allocations are unpredictable, and insufficient for the needs of the sector; furthermore, the funding that is made available is heavily skewed towards capital expenditure over recurrent, from which maintenance is largely funded. The situation is further complicated by the fact that the full amount allocated in the budget is not released, and the funding is received late in the financial year. This places severe restrictions on the ability of the sector to implement long term plans.

Whilst a RAMS can be developed and implemented to some extent by consultants through donor funding, the sustainability of a RAMS approach depends on government’s understanding of the benefits of RAMS, the allocation of enough funds for the purchase and maintenance of equipment, regular data collection surveys, provision of personnel for a dedicated RAMS unit, and the provision of appropriate IT hardware and software licences. Such budget streams have been requested but have not yet fully materialized. Successful RAMS implementation requires a significant cultural change and a shift of focus to road user satisfaction and continual improvement processes.

Technical Challenges

At inception of the RAMS programme in 2009, there was limited in-house technical capacity to support a RAMS. The issues encountered were:

- No equipment readily available for road data collection surveys; funds were scarce for large, expensive items such as survey vehicles, and any smaller items available were scattered among different departments of the Agency. Any previous training on data collection had long been forgotten from lack of use.
- Limited information on road inventory. There were documents giving text-based descriptions of routes and their lengths, but these often varied between documents (and even within the same document). For example, the reported length of Federal road network can vary between about 34,500 and 36,000 km depending on which section of the same document consulted.
- No planned routine surveys of road or bridge condition, consequently there was little information on the condition of the assets. Engineers in individual States would carry out a driven inspection of
their routes when required, but the results were rarely reported in a form that could be used in maintenance planning.

- Nigeria has a large road network which was quite poorly mapped. Road maps are available, but they are not geographically exact, and route names and locations vary from one to the other. To function properly, a RAMS requires a road network representation that is as accurate as possible from a spatial perspective, i.e. the map shows the ‘real world’ shape and length of the route rather than a schematic representation. This accuracy is required in order that data collected from surveys can be assigned correctly to the sections of road. No such geo-referenced map of the Federal road network existed at the time.

Despite having addressed the initial challenges to a large extent, up-to-date and comprehensive information required to develop prioritized maintenance plans for the network is often missing. Detailed information required to plan for the different types of maintenance activity carried out in Nigeria, the intervention criteria used, unit costs, mobilization costs, etc. There is an ongoing initiative to adopt service level-based route categorization and associated development and implementation of key performance and value-for-money indicators linked to the different service levels. Thus, a fully operational RAMS enabled to generate prioritized forward works programmes and provide budget estimates and analysis of the impact on network performance of different levels of funding is yet to be fully realized. The RAMS process requires regular surveys of the road network to obtain up-to-date information on the condition of the roads and the levels of traffic on them (as this is the main cause of deterioration). This information is then fed into software tools that analyze the data and develop optimum maintenance plans for the network.

5 GOING FORWARD

Sustainability of FERMA’s RAMS programme is at the core achieving best value in maintenance planning and managing the Nigeria’s Federal road network. The Agency is mindful of instances of software that has been procured through development programme in other countries, and then remains unused after the programme comes to an end because the local agencies cannot afford the on-going licence fees. NIAF has provided a free perpetual licence for Nigeria to build a sustainable RAMS process for the future. The current phase of the programme involves the implementation of the system within the Agency and the embedment of processes and activities that will ensure that the RAMS is sustainable in the longer term. Requisite organisational restructuring and streamlining of processes to align with asset management approach is being considered. A team of users has already been trained in the use of the software on desktop computers whilst working towards a web enabled software so that users throughout Nigeria have direct access to the system. A survey regime that is appropriate for Nigeria has been proposed and intended to provide a steady supply of the up-to-date road and bridge condition data that is essential to inform good maintenance management.

6 CONCLUSIONS

Sustainable deployment of RAMS in maintenance planning and managing the Federal road network in Nigeria requires the commitment, involvement and the buy-in of Agency’s senior managers and awareness of the benefits of RAMS over ad hoc reactive maintenance. Successful RAMS implementation requires a significant cultural change and a shift of focus to road user satisfaction and continual improvement processes. RAMS needs to be steered by a vision, a policy and a strategy, and implementation and embedment of a set of KPIs to monitor the performance of the network. In addition, the establishment of requisite organizational structures with clear roles and responsibilities, staff training, capacity building and knowledge transfer underpin successful implementation. Staff provision include IT support and maintenance services, staff (users and technical support staff) with IT skills. Funding streams that support RAMS processes are equally essential. Stable and predictable funding streams to support medium/long term planning and maintenance strategies implementation, and funding for sustainable data collection processes (survey cycles, defects, equipment maintenance/ renewals, etc.). Not all of these are in place and thus pose potential risks to the programme.

7 REFERENCE
8 ACKNOWLEDGEMENT

The support and input from DFID-funded NIAF programme is gratefully acknowledged.
KEYWORDS:
Road Debris, Roadkill, Road Safety, Prototype, Road Maintenance

ABSTRACT:
There are no obvious standards about how to load goods on vehicles, especially pickup trucks, and 1-ton truck. 1-ton truck is the most prevailing vehicles in Korea. Although there are clear restrictions on the height and/or weight of them in current laws, there are no obvious standards about how to load or bind goods on vehicles such as trucks in South Korea. Unfortunately, a clear definition of such “road debris” does not exist in any official document. It usually includes fragments derived from traffic accidents, flat tire, roadkills, and trashes blown onto the road by wind, and so on. This paper introduces the prototype, explains the results of tests that have been conducted to assess its performance in an experimental environment, and then discusses ways to realize its further advancement. For the purpose of the study, CATIA (Computer Aided Three-Dimensional Interactive Application) was used. The software program allows the implementation, modification and control of the entire design and manufacturing process of industrial product models. It is adopted widely in the aerospace engineering and automobile industries as well as in related disciplines. Various road debris dummies are employed to test rate of success of collect using a prototype. The tests conducted so far showed better removal success rates for relatively larger and heavier objects or very light objects among the selected dummies. In the future, more tests will be conducted with debris placed at diverse locations on the road. Ultimately, the newly developed vehicle will be actually used for the removal of road debris starting at the end of the year 2019.
1 INTRODUCTION

Although there are clear restrictions on the height and/or weight of goods in current laws, there are no obvious standards about how to load or bind them on vehicles such as trucks in South Korea. The police address this issue with the state of Korean Road Traffic Act, Article 39 and clause 4. It states that “the drivers of all vehicles shall take necessary actions to firmly fix goods on their vehicles by putting a cover on it or binding it, so that they do not fall off the vehicle during driving”. Thus, if goods are not properly loaded on vehicles, there are occurrence of objects falling from vehicles, especially small pickup trucks and 1-ton truck on the road for various reasons during travelling. Particularly, 1-ton truck is the most prevailing truck type in Korea.

As illustrated in Figure 1, traffic accidents can be produce a lot of fragment and remnants. We can consider them as road debris. Unfortunately, a clear definition of such “road debris” does not exist in any official document. It usually includes fragments derived from traffic accidents, flat tire, roadkills, and trashes blown onto the road by wind, and so on.

![Fig. 1 Various Road Debris on Korean Roadways](image)

Such road debris on the road may cause traffic delay or congestion by obstructing the traffic flow. More seriously, it can be a significant threat to safety. Currently, public road workers are in charge of removing debris on the road as quickly as possible. However, the number of public road workers for national highways which are the major arterial roads in Korea, declined to 500 people in 2013 from 700 people in 2004. During that time, there was no additional employment. Such reduction was resulted from death or injury from accidents during their work. It is larger than the decrease due to normal retirement.

The situation is no better for Korean express highways. Vehicles that are likely to drop objects on the road due to improper loading of goods are found out in the front of and/or around the tollgate. The number of such cases was approximately 96,000 in 2013 and 93,000 in 2014. A wide variety of road debris are found out on highways, including pieces of wood, pebbles, sand, iron objects, plywood, and the like. Over the past five years, 232 accidents were caused by road debris on highways to kill one person and injured 79 people. In other words, on average, 46 accidents are caused by road debris annually. The number of traffic accidents caused by road debris or roadkill has neither risen nor fallen over the past five years.

A serious problem related to such situations is that the number of deaths of public road workers who perform the elimination of road debris has been increasing. This has brought about a paradigm shift in Korean road management policy from manpower-oriented operation to automation. As a visible plan and measure for such change, the development of a vehicle that can automatically remove debris from roads began in 2015 as a national R&D project, with the aim to complete a prototype in 2017. Equipment is used to pick up road debris in the United States, but the
The prototype developed in Korea is different in every aspect, including the method of pickup, type of removed objects, and input of manpower, as it was developed in compliance with Korea’s Road Traffic Act and Framework Act on Vehicle Safety. Currently, evaluation of its performance is underway in tests on actual roads. The guidelines for its operation on actual roads in the future have also begun to be developed. This paper introduces the prototype (ROBOS), explains the results of tests that have been conducted to assess its performance in an experimental environment, and then discusses ways to realize its further advancement.

2. LITERATURE REVIEW AND TECHNOLOGY TRENDS

Patent applications regarding road debris removal technology revealed a total of 13 such applications of which 7 cases were available for citation in South Korea. The corresponding number overseas was 354 in total. Four of them were found to be citable. Investigation of the applications’ details showed that very few technology solutions existed which specifically addressed road debris removal, and that the majority of the applications simply utilized conventional cleaning equipment or machines to collect the road debris (Table 1).

<table>
<thead>
<tr>
<th>Patent name</th>
<th>Application year</th>
<th>Patent drawings</th>
<th>Applicant</th>
</tr>
</thead>
<tbody>
<tr>
<td>The road surface rubbish collect equipment</td>
<td>2013</td>
<td></td>
<td>KISUNG Engineering, South Korea</td>
</tr>
<tr>
<td>ROAD DEBRIS REMOVAL EQUIPMENT</td>
<td>2009</td>
<td></td>
<td>Korean Expressway Corporation, South Korea</td>
</tr>
<tr>
<td>ROAD WORK REFUSE COLLECTION APPARTUS</td>
<td>2011</td>
<td></td>
<td>YOUNGSIK KIM, South Korea</td>
</tr>
<tr>
<td>Multi-functional street cleaning vehicle</td>
<td>2008</td>
<td></td>
<td>RETECH Technology &amp; Seoul Metropolitan Facilities Management Corporation, South Korea</td>
</tr>
<tr>
<td>Road surface removing machine</td>
<td>1979</td>
<td></td>
<td>Allied Steel &amp; Tractor Products, Inc. Solon, US</td>
</tr>
<tr>
<td>Vehicle for collecting debris from a road</td>
<td>1995</td>
<td></td>
<td>David Tolmachoff, US</td>
</tr>
</tbody>
</table>

The AAA Foundation for Traffic Safety (2004) investigated danger associated with debris on North American roads (2004). The Colorado Department of Transportation (CDOT) carried out field tests on the Gator Getter™, a high speed debris remover instrument developed by CDOT. Valdes-Vasquez et al. (2014) conducted evaluation of the performance of the instrument. However, it was found that this is not appropriate to be used in Korea, in consideration of the road management method and relevant laws and regulations. In 2016, we proposed specific directions for a design concept in the development of road debris remover equipment customized for use on Korean roads (Yang et al, 2016). As of 2018, the prototype has already been developed and the rate of successful debris removal (%) is tested with diverse road debris dummies in an experimental road environment.

3. LAYOUT DESIGN AND PROTOTYPE

Based on the results of the surveys, Table 2 lists considerations to be made while designing and developing the instrument for removing road debris. A truck capable of carrying a 2.5 ton load was selected for the design development taking into account the preferred appropriate amount of debris per removal around 1 ton. Additionally, the vehicle’s construct and the types of debris items to be picked up were examined and then incorporated into the layout. The latter
stages of the development will also incorporate travel speed of the vehicle, the total weight of debris per removal plus other factors.

<table>
<thead>
<tr>
<th>Considerations</th>
<th>Design Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vehicle</td>
<td>Over 2.5 ton long-axis truck</td>
</tr>
<tr>
<td>Operating Speed</td>
<td>Over 15km/hr</td>
</tr>
<tr>
<td>Allowable Debris Weight by Type</td>
<td>· Ordinary waste: max. 5 kg (approx.)</td>
</tr>
<tr>
<td></td>
<td>· Carcass: 11 kg (approx.)</td>
</tr>
<tr>
<td>Collection Capacity</td>
<td>Approximately 2 ton</td>
</tr>
<tr>
<td>Vehicle</td>
<td>Must be able to:</td>
</tr>
<tr>
<td></td>
<td>· Collect both carcass and ordinary waste</td>
</tr>
<tr>
<td></td>
<td>· Separate carcass from non-carcass debris</td>
</tr>
<tr>
<td></td>
<td>· Satisfy relevant vehicle safety standards</td>
</tr>
<tr>
<td>Debris Type</td>
<td>· Sand and pebble</td>
</tr>
<tr>
<td></td>
<td>· Carcass, and debris remaining after auto accidents</td>
</tr>
<tr>
<td></td>
<td>· Any and all debris obstructing traffic flow (plastics, wood chips, boxes)</td>
</tr>
</tbody>
</table>

For the purpose of the study, CATIA (Computer Aided Three-Dimensional Interactive Application) was used. The software program allows the implementation, modification and control of the entire design and manufacturing process of industrial product models. It is adopted widely in the aerospace engineering and automobile industries as well as in related disciplines.

Fig. 2 illustrates the layout of the truck to pick up debris. In consideration of the diversity of debris, the vehicle’s frame height was raised by about 30 cm so as to allow the mounting of the removing equipment underneath the frame. This design reflects difficulties securing vehicle safety certification which can often be encountered with front-mount applications. Also, difficulties picking up debris were considered with rear-mount apparatuses. Further, front-mount designs would limit the types of debris due to the set amount thereof to be removed. Significantly poorer vehicle safety during removal operation carried out at higher travel speeds was another disadvantage associated with front-mount designs. The rotating brushes at the front in Fig. 2 draw the debris in and push it underneath the frame. Once underneath, the debris will be transported onto the conveyor belt via underside collection tool and then to the freight space. For hygiene purposes, the design provides separate loading space for roadkill and ordinary debris items. A front sweeper pulls debris outside the width of the vehicle toward the center of the vehicle. The prototype was developed based on the above-mentioned design drawing. In Fig. 3 shows that the prototype was developed based on the final design.
4. ROAD TEST RESULTS

Various road debris dummies are employed to test rate of success of collect using ROBOS. The followings were used for the tests.
- Roadkill dummy (2 types)
- Plastic boxes divided into large, medium, and small ones, according to their size
- Brick
- Wood
- Plastic bottle
- Cans divided into medium and small ones according to their size
- Shoes
- 16-inch tire

The road debris dummies used for the tests are the types of debris frequently seen on actual roads. They were selected based on the opinions of road agencies and relevant workers in charge of this road maintenance. Access to the road for the test by other vehicles than the test vehicle was blocked to ensure accurate calculation of the successful removal rate as well as safe environment as depicted in Fig. 4. After the road had been blocked, debris was randomly placed on the road, and the tests were conducted repeatedly. In addition, the location of the debris was set at the center of the road, that is, the center of the direction for the vehicle’s driving, for convenience of removal. Fig. 4 shows the test road.

The vehicle travelled 10 km on the road at a speed of 15 km/hr. Collection of each type of debris was performed a total five times under the same conditions to measure whether the collection was successful. The results of the tests are shown in Table 3.

<table>
<thead>
<tr>
<th>Debris Type</th>
<th>Number of Test Conducted</th>
<th>Results</th>
<th>Rate of success (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two types of roadkills</td>
<td>5 times</td>
<td>Succeeded 5 times</td>
<td>100</td>
</tr>
<tr>
<td>Tire</td>
<td>5 times</td>
<td>Succeeded 5 times</td>
<td>100</td>
</tr>
<tr>
<td>Plastic box L</td>
<td>5 times</td>
<td>Succeeded 5 times</td>
<td>100</td>
</tr>
<tr>
<td>Plastic box M</td>
<td>5 times</td>
<td>Succeeded 2 times</td>
<td>40</td>
</tr>
<tr>
<td>Plastic box S</td>
<td>5 times</td>
<td>No success</td>
<td>0</td>
</tr>
<tr>
<td>Brick</td>
<td>5 times</td>
<td>Succeeded 4 times</td>
<td>80</td>
</tr>
<tr>
<td>Wood L</td>
<td>5 times</td>
<td>Succeeded 2 times</td>
<td>100</td>
</tr>
<tr>
<td>Wood S</td>
<td>5 times</td>
<td>No success</td>
<td>0</td>
</tr>
<tr>
<td>Shoes</td>
<td>5 times</td>
<td>Succeeded 2 times</td>
<td>100</td>
</tr>
<tr>
<td>Plastic bottle</td>
<td>5 times</td>
<td>No success</td>
<td>0</td>
</tr>
<tr>
<td>Aluminum can M</td>
<td>5 times</td>
<td>No success</td>
<td>0</td>
</tr>
<tr>
<td>Aluminum can S</td>
<td>5 times</td>
<td>No success</td>
<td>0</td>
</tr>
</tbody>
</table>

The debris whose removal rate was 50% or higher were the two dead animal dummies, tire, plastic box L (large), wood L (large), and shoes. Except for plastic box M (medium), the other debris showed a very low successful removal rate. The test results showed that the removal rate of road debris dummies with large volume and heavy weight was significantly high, but the rate for smaller and lighter debris dummies was lower. Such results seems to be related to the capacity and power of the vacuum pump, so it is thought that the performance of the vacuum pump needs to be improved as much as possible in consideration of the type of vehicle. Accordingly, the capacity and power of the vacuum removal instrument were improved, and then the test was conducted again on the same test road with the same road debris dummies and in the same road environment. The results are presented in Table 4.
Table 4. Test Results for Each Road Debris Dummy (2nd )

<table>
<thead>
<tr>
<th>Debris Type</th>
<th>Number of Test Conducted</th>
<th>Results</th>
<th>Rate of success (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two types of roadkills</td>
<td></td>
<td>Succeeded 5 times</td>
<td>100</td>
</tr>
<tr>
<td>Tire</td>
<td></td>
<td>Succeeded 5 times</td>
<td>100</td>
</tr>
<tr>
<td>Plastic box L</td>
<td></td>
<td>Succeeded 5 times</td>
<td>100</td>
</tr>
<tr>
<td>Plastic box M</td>
<td></td>
<td>Succeeded 3 times</td>
<td>40</td>
</tr>
<tr>
<td>Plastic box S</td>
<td></td>
<td>Succeeded 2 times</td>
<td>40</td>
</tr>
<tr>
<td>Brick</td>
<td>5 times</td>
<td>Succeeded 4 times</td>
<td>80</td>
</tr>
<tr>
<td>Wood L</td>
<td></td>
<td>Succeeded 5 times</td>
<td>100</td>
</tr>
<tr>
<td>Wood S</td>
<td></td>
<td>Succeeded 3 times</td>
<td>60</td>
</tr>
<tr>
<td>Shoes</td>
<td></td>
<td>Succeeded 5 times</td>
<td>100</td>
</tr>
<tr>
<td>Plastic bottle</td>
<td></td>
<td>Succeeded 3 times</td>
<td>60</td>
</tr>
<tr>
<td>Aluminum can M</td>
<td></td>
<td>Succeeded 4 times</td>
<td>80</td>
</tr>
<tr>
<td>Aluminum can S</td>
<td></td>
<td>Succeeded 4 times</td>
<td>80</td>
</tr>
</tbody>
</table>

As seen in the results of the second test conducted after the improvement, the removal rate was 50% or higher for all debris except for two of them. However, for those two, the success rate increased to 40% in the second test from 0% in the first test. Notably, the rate for can M (medium) and can S (small) with a very small volume and weight increased to 80% in the second test from 0% in the first test, proving that the performance enhancement of the vacuum instrument was effective. Currently, a test of removing debris fallen from various locations on the road is underway. For example, in the current test, debris fallen on the median strip or top of the road are picked up. The same debris dummies as those used in the first and second tests are being used in the ongoing test, but the running speed of the Robos during the debris removal (5 km/hr~15 km/hr) is adjusted according to the location of the debris. After completing all tests in the test environment, the debris removal vehicle will be used on actual roads.

5. CONCLUSION

Despite the government’s efforts to control overloaded vehicles on the road in South Korea, the number of road debris-induced accidents is increasing every year. Such accidents being an extremely likely cause of massive accidents, preventive measures should be established to protect the drivers as well as the highway maintenance crews against safety hazards. As a detailed action plan to adapt to the paradigm shift in Korea’s road management policy, a vehicle that can automatically remove road debris began to be developed in 2015, as a national R&D project. Then, in 2017, a prototype was completed. Currently, tests on the prototype are being conducted with various debris dummies in a restricted road environment. The tests conducted so far showed better removal success rates for relatively larger and heavier objects or very light objects among the selected dummies. In the future, more tests will be conducted with debris placed at diverse locations on the road. Ultimately, the newly developed vehicle will be actually used for the removal of road debris starting at the end of the year 2019.

REFERENCES
1. AAA Foundation for Traffic Safety. (2004), The safety impact of vehicle related road debris
2. Colorado Department of Transportation. (2014), CDOT rapid debris removal research project, DReport No. CDOT-2014-9
Sustainable transportation for cities: A comparison between Seoul, Korea and Tokyo, Japan

KEYWORDS:
Transportation assessment, Comparison of city, Transportation strategies, Traffic System Management, Transportation Demand Management

ABSTRACT:
The performance of transportation strategies and policies for Seoul, South Korea, and Tokyo, Japan, was compared. Evaluating transportation services and systems provides valuable information for management. Especially comparing similar cities might help benchmark each other. Diverse factors were considered to evaluate transportation performance. Many types of socio-economic measures provided critical information. Detailed metrics from transportation modes such as travel times, mode sharing rate, and transit passengers were used to determine effectiveness. The comparison and evaluation provide material for a new policy to seek better system performance. Both cities can learn from each other about the corresponding best practices and performance metrics.
Sustainable transportation for cities:
A comparison between Seoul, Korea and Tokyo, Japan

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1 INTRODUCTION

Evaluating and comparing the performance of transport systems and services between cities could provide information that reconfirms their feasibility and can be used to plan enhancements and expansion in the future. In addition, by comparing cities each other, the merits and drawbacks existing in each city could be clarified to address critical issues in an appropriate manner.

This study compared the transportation systems of two cities: Tokyo, Japan and Seoul, Korea. Both cities have transportation infrastructures and operate services that are well organized. Diverse aspects in transportation would be analyzed for their comparison and evaluation. By analyzing the status, the transportation systems in these cities, realistic and useful experiences regarding the management and operation the systems could be shared, possibly enhancing their effectiveness. Questions below might be possible from people living or working in big cities, including:

- Are the citizens satisfied with the existing transport system?
- Is the existing transport system suitable to accommodate demands?
- Are the transport modes efficiently connected and collaborated?
- Which strategies might be possible to remedy traffic congestion issues in a satisfactory manner?
- What are policies are needed to reduce significant traffic accidents?

Answers on the questions throughout comparison would help enhance performances of the existing services in two cities. Categorized strategies on recent traffic issues also show here.

2. LITERATURE REVIEW

Morichi & Acharya addresses the challenges, that public transport, a sustainable mode of mobility, is subjected to a vicious cycle of poor service, decreasing ridership and lower investment. They provide insightful analysis and novel viewpoints for road and public transport policy including special development, funding and financing. According to their discussion, the basic characteristics of Asian megacities and the nature of urban transport problems seem to be different from those in megacities from other parts of the world. Without considering such typical characteristics, appropriate policies for sustainable transport development cannot be formulated, let alone effectively implemented. Coordinated or integrated transport system has been remained one of the major transport policy agenda since long ago in developed countries. (Morichi & Acharya, 2012).

Ieda analyze the experiences (successes and failures) of cities including Tokyo and Seoul toward building sustainable transport systems for people. He discusses the development of Tokyo rail system and distinctive features. He pointed out that Tokyo’s public transport has been based on the private-oriented management of operators. This resulted in positive outcomes such as the efficient, cost-minded management, and rail development that were commercially combined with various regional developments (Ieda, 2010). International comparison about transportation policy and practice draw several insightful results.

The Asian region is rapidly developing its own archetype of urban traffic dysfunction: the Bangkok metropolitan region. The similarities between Los Angeles and Bangkok today have some deeper historical significance (Kenworthy, 2017). Marcotullio & Lee compare the development of urban transportation systems with other challenges to demonstrate a different, compressed form of the transition (Marcotullio & Lee, 2003).
Waley outlines three different types of comparative urbanism and sets out a basic framework for the study of urban change in the larger cities of China and Japan. His argument is that the close relationship between the state and capital in the two countries has conditioned the rapid and dynamic nature of urban change (Waley, 2012). Understanding Tokyo and Seoul necessitates a different conception of the world system from the globalist version of the world city argument (Child Hill & Kim, 2000). The comparison sometime covers CO2 emissions and environmental indicators (Lee & Van de Meene, 2013), socio-economic developmental indicators and historical trajectories in urban development (Huang, et al., 2007). The “unrestrained motorisation” model is contrasted with the experiences of wealthier Seoul, Singapore, Hong Kong and Tokyo, which have all restrained and slowed the pace of motorisation to some extent and enhanced the role of public transport. In all four cities, 1990 levels of motorisation and vehicle use were low relative to their levels of income (Barter, 1999).

A basic plan in year 2000 has built by considering better policy (Kim and Rim, 2000). A big picture of Korean federal government has included master plan of public transportation for Seoul (Urban transportation division, 2006). It has asserted that Seoul’s bus reform and public transportation has decisive and direct influence Lee become a president in Korea. When a mayor of big cities in America has done a great accomplishment in traffic matters, he or she might obtain an opportunity becoming a president of USA (Marshall, 2012). A few literature has mentioned about success of Seoul public transportation and bus reform (Seoul public transportation, 2016, Seoul special city, 2006, and Lee & Lee, 2013). The literature had pointed out the reasons of successful public transportation performance in detail. Lee, G has explained the whole history of Seoul transportation with big data. The analysis has brought background of bus reform in 2006, then predicted the future of Seoul transportation. Also, website of Seoul TOPIS has provided merits and demerits of public transportation in Seoul.

3. GENERAL TRANSPORTATION CONDITIONS

Table 1 compares several factors that may affect transportation in both Seoul and Tokyo. Regarding area, population, and the number of vehicles, all these factors are slightly greater in Tokyo than in Seoul. In terms of area, for example, Tokyo is about four times larger than Seoul. In addition, Tokyo's roads and rails have heavy use. In contrast, Seoul has many more buses and taxis than Tokyo.

Table 1 Factors affecting the transportation systems in Seoul, Korea and Tokyo, Japan

<table>
<thead>
<tr>
<th>FACTORS</th>
<th>SEOUL</th>
<th>TOKYO</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area (km²)</td>
<td>605¹</td>
<td>2,188³</td>
</tr>
<tr>
<td>Population</td>
<td>10,000,000¹</td>
<td>14,000,000³</td>
</tr>
<tr>
<td>Vehicle</td>
<td>3,000,000¹</td>
<td>4,413,094³</td>
</tr>
<tr>
<td>Road Distance (km)</td>
<td>8,215¹</td>
<td>24,498³</td>
</tr>
<tr>
<td>Rail Distance (km)</td>
<td>331¹</td>
<td>1,054³</td>
</tr>
<tr>
<td>No. of Buses</td>
<td>7,500²</td>
<td>1500³</td>
</tr>
<tr>
<td>No. of Taxis</td>
<td>72,000²</td>
<td>60,000³</td>
</tr>
<tr>
<td>No. of Accidents</td>
<td>40,792²</td>
<td>37,184²</td>
</tr>
<tr>
<td>Fatalities</td>
<td>400²</td>
<td>172²</td>
</tr>
<tr>
<td>Wounded</td>
<td>57,345²</td>
<td>43,212²</td>
</tr>
</tbody>
</table>

2. Seoul transport data, Seoul Metropolitan Government, 2017
3. Tokyo Metropolitan Government, Bureau of Construction, 2017
4. Tokyo Metropolitan Police Department, 2017

Issues regarding accidents seem to be a serious problem in Seoul compared to Tokyo. If using population and road distance to estimate the number of road accidents in these two cities, it is easily apparent that Seoul has worse statistics than Tokyo. Seoul has three times more fatalities than does Tokyo, among the worst level of statistics for member countries of the Organization for Economic Co-operation and Development (OECD).

As shown in Table 2, while Tokyo mostly relies on railways, Seoul has coordinated the bus and rail systems. In other words, a few modes are available to select along routes in Seoul. For example, passengers
who dislike traveling in underground subways, without a view outside, can choose a bus to their destination or else can transfer in the middle of their route. Such various modes seem accommodate diversity reflecting preference and inclination. In Tokyo, rail passengers ride on bikes from home to the station. The bike and rail transfer system in that city helps travelers to access their homes in a more convenient and faster way than in Seoul.

Table 2 General comparisons of the transportation systems in Tokyo and Seoul

<table>
<thead>
<tr>
<th>TOKYO</th>
<th>SEOUL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rail oriented</td>
<td>Coordination bet. railway and bus</td>
</tr>
<tr>
<td>Rail and bike transfers</td>
<td>Bus and rail transfers</td>
</tr>
<tr>
<td>Sophisticated traffic behavior</td>
<td>Poor (immature) traffic behavior</td>
</tr>
<tr>
<td>Stagnant for car growth</td>
<td>Car ownership is still growing</td>
</tr>
<tr>
<td>Led by industry</td>
<td>Led by government</td>
</tr>
<tr>
<td>Stable driving circumstances</td>
<td>Risky driving circumstances</td>
</tr>
<tr>
<td>Excellent parking system</td>
<td>Illegal parking is prevalent</td>
</tr>
<tr>
<td>High transit fares</td>
<td>Low transit fares</td>
</tr>
<tr>
<td>Hidden transit subsidy</td>
<td>Open transit subsidy</td>
</tr>
</tbody>
</table>

While Tokyo has enacted laws to regulate good traffic behavior, known as traffic manners, traffic laws to regulate the behavior of Seoul's transportation culture have not been appropriately implemented. Immature traffic manners are prevalent in Seoul and might be closely related to unexpected accidents and traffic jams on the roads. While private vehicle ownership still is growing rapidly in Seoul, it has stagnated in Tokyo.

In Seoul, transportation matters sometimes tend to be dealt with based on politics, and not with policies based on the opinions of its citizens. Therefore, in Seoul, the government mostly leads to resolve transportation issues. In contrast, Tokyo's government officials communicate closely with the car industries to obtain and share the benefits in solving traffic issues. Regarding policy implementation, both cities have done well in building and operating infrastructure in a proper way. However, in specifically managing traffic policies, Tokyo has been successful in its strict enforcement of traffic regulations, and Seoul has had comparatively weaker enforcement. Due to the differences in policy execution, the streets in Seoul have dangerous conditions for driving and walking as compared to Tokyo.

Drivers in Seoul usually feel that it is difficult to find a parking space. The lack of parking lots means that drivers are on the roads for a longer period, emitting toxic materials. In Tokyo, it is not that hard to find a parking lot because there is plenty of space to park around the city. However, Tokyo has a 'parking lot requisition' policy, in which a parking space must have been assured at home or near its vicinity before buying a car. That policy has been very efficient in taking care of the parking demands in the Tokyo metropolitan area. Conversely, illegal parking that predominates in Seoul has been brought about by a shortage of parking spaces around residential and commercial areas.

Regarding transit fares, they are low in Seoul compared to Tokyo. In fact, transit fares in Seoul have been evaluated as one of the lowest in the world. A significant benefit of these low fares is that more passengers are persuaded to use sustainable means of transportation, such as rail or buses. There are no additional transfer charges between modes of transit, which might persuade drivers using private vehicles decide to use the transit system instead. Overall, about 2 US dollars would be sufficient to travel around Seoul metropolitan area. In Tokyo, the fare is estimated exactly, based on distance and stations travelled. The total charges have become excessively high when compared to Seoul. However, subsidies to compensate for the high fares are supported from work places. Both cities also have an effective pension system to the senior citizens which compensate for transit fares.

4. COMPARISON OF MAJOR ISSUES

4.1 General Conditions and Traffic Congestion

Recently, both cities have experienced extreme traffic congestion, caused by many vehicles along most roads around the metropolitan region, especially during peak hours. As shown in Table 3, congestion in Seoul has worsened year by year, probably due to an increase in vehicle ownership. However, congestion in
Tokyo tends to be stagnant, likely due to no increase in car ownership in Tokyo.

Table 3 Congestion conditions in Seoul and Tokyo

<table>
<thead>
<tr>
<th></th>
<th>SEUL</th>
<th>TOKYO</th>
</tr>
</thead>
<tbody>
<tr>
<td>Range</td>
<td>Multiple districts</td>
<td>Central Business District (CBD)</td>
</tr>
<tr>
<td>Causes</td>
<td>Demand &gt; Supply</td>
<td>Excessive vehicles into the CBD</td>
</tr>
<tr>
<td>Trend</td>
<td>Worsening</td>
<td>Remains the same</td>
</tr>
<tr>
<td>Travel time</td>
<td>Exclusive bus lanes in operation</td>
<td>Best among member cities of OECD countries</td>
</tr>
<tr>
<td>Treatment</td>
<td>Irregular</td>
<td>Sustainable</td>
</tr>
</tbody>
</table>

Meanwhile, according to a survey that estimated congestion costs in Seoul, these costs have increased by about 50% over the last decade, with an estimated 10 billion US dollars (USD) in 2016. Abruptly increasing costs have hurt the city financially as well as worsened the conditions for mobility. For the total percentage of travel modes used in these two cities, about 70% of urban travelers in both cities use public transportation. This means that transit has an important role in handling transportation demands in these cities, as shown in Table 4.

Table 4 Congestion costs and commuting time in Seoul and Tokyo

<table>
<thead>
<tr>
<th></th>
<th>SEUL</th>
<th>TOKYO</th>
</tr>
</thead>
<tbody>
<tr>
<td>Congestion cost</td>
<td>10 billion US dollars¹</td>
<td>N/A</td>
</tr>
<tr>
<td>Modal share rate for transit</td>
<td>68%¹</td>
<td>70%¹</td>
</tr>
<tr>
<td>Commuting time</td>
<td>58 min²</td>
<td>40 min²</td>
</tr>
</tbody>
</table>


Regarding the area and population of these two cities, the modal share rate for transit seems to have reached a peak for both. Similarly, transit modal share rates in other major cities are at around 70%. This means that other modes of travelling are used 30% of the time. By the way, average commuting time in Tokyo shows faster time than the one in Seoul.

To overcome serious traffic congestion in Seoul, a policy has been implemented to assign exclusive bus lanes along the main corridors; as a result, travel time has been reduced significantly for transit riders as well as for drivers in private vehicles. Therefore, executing sustainable transportation policies are critical to handle appropriately the increasing demand in Seoul. The general traffic situation in two cities looks similar right now; however, the choices made in managing the infrastructure and implementing policies could bring about many different outcomes.

4.2 Safety

Accidents have been decreasing in recent years for both cities, as shown in Table 5. However, the frequency of accidents in terms of road length, population, and vehicles vary for both cities. While five accidents per kilometer a year have occurred in Seoul, about 1.5 accidents per kilometer per year has occurred in Tokyo. This indicates Tokyo is three times safer than in Seoul. Regarding fatalities in accidents, both cities have similar outcomes. Fatalities in Seoul have occurred three times more than in Tokyo. Data on people wounded during accidents provide almost identical results as for fatalities, with Tokyo being much safer. When the total wounded was estimated in terms of distance (in kilometers), Tokyo had much lower numbers than Seoul. Therefore, even though Tokyo has a larger population and greater area than Seoul, it could be said that Tokyo is much superior to Seoul in terms of traffic safety.
Table 5 Statistics regarding traffic accidents

<table>
<thead>
<tr>
<th></th>
<th>SEOUL</th>
<th>TOKYO</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of accidents</td>
<td>40,792¹</td>
<td>37,184²</td>
</tr>
<tr>
<td>Accidents / km</td>
<td>4.97¹</td>
<td>1.52²</td>
</tr>
<tr>
<td>Fatalities</td>
<td>400¹</td>
<td>172²</td>
</tr>
<tr>
<td>Fatalities / population, 100k</td>
<td>4¹</td>
<td>1.23²</td>
</tr>
<tr>
<td>Fatalities / vehicles, 10k</td>
<td>1.3¹</td>
<td>0.39²</td>
</tr>
<tr>
<td>Deaths / km</td>
<td>0.05¹</td>
<td>0.01²</td>
</tr>
<tr>
<td>Wounded</td>
<td>57,345¹</td>
<td>43,212²</td>
</tr>
<tr>
<td>Wounded / km</td>
<td>6.98¹</td>
<td>1.76²</td>
</tr>
</tbody>
</table>

¹. Seoul transport data, Seoul Metropolitan Government, 2017
². Tokyo Metropolitan Police Department, 2017

There are many reasons that traffic accidents occur in a city. In fact, big cities might not be able to avoid traffic accidents due to an excessive number of vehicles. Seoul and Tokyo have been experiencing accidents every day on their roads. Beyond the number of vehicles, traffic jams tend to induce accidents, and accidents are followed by traffic jams, causing a vicious cycle to occur.

Although the two cities have been under similar conditions, Tokyo has shown a greater ability to handle traffic jams and accidents than has Seoul, as shown in Table 6. These contrasting results might be due to difference in the traffic manners or transportation culture of each city. It is highly recommended that appropriate strategies and policies be developed for traffic accidents, based on current road conditions and accident status. Moreover, being able to listen to various opinions from the public has a crucial role in successfully managing accidents and traffic jams. In Seoul, the federal police agency, which is separate from Seoul's government, has most of the responsibilities in handling accidents issues. Usually, however, it is slightly more effective when the local government handles issues involving traffic accidents. Because traffic accidents involve many other city departments in Seoul, it is necessary for these departments to collaborate with one another inside City Hall.

Table 6 Handling traffic accidents in Seoul and Tokyo

<table>
<thead>
<tr>
<th></th>
<th>SEOUL</th>
<th>TOKYO</th>
</tr>
</thead>
<tbody>
<tr>
<td>Accidents</td>
<td>Have decreased</td>
<td>Have decreased</td>
</tr>
<tr>
<td>Causes</td>
<td>Congestion / traffic manners</td>
<td>Congestion</td>
</tr>
<tr>
<td>Responsibility</td>
<td>Federal police</td>
<td>Police and local authorities</td>
</tr>
<tr>
<td>Policy</td>
<td>Focus on death reduction</td>
<td>Concern regarding senior citizens</td>
</tr>
<tr>
<td>Infrastructure</td>
<td>Improved</td>
<td>Stable, but old</td>
</tr>
</tbody>
</table>

While Seoul has focused on reducing accidents, Tokyo focused on safety due to their aging infrastructure. In Seoul, the rate of traffic accidents has decreased due to efficient safety policies and the enhancement of safety facilities, with a goal of zero accidents. Over the last few decades, Tokyo has strongly enforced all types of traffic violations; overall, this city has a safe and convenient traffic atmosphere. However, Tokyo's aging infrastructure might be a barrier to keeping safe road conditions.

In Seoul, speeding may be key factor that has threatened a safe traffic environment. Mechanical and electrical improvements regarding the quality of vehicles have resulted in allowing over-speeding on streets. Drunk and reckless driving habits also hurt normal road management, and thus produce accidents in Seoul. In addition, illegally parked vehicles have resulted in uncomfortable walking conditions, causing accidents unnecessarily along the streets. In Tokyo, however, vehicle speeds seem to stay within the restricted speed range, resulting in a low probability of conflicts between vehicles and pedestrians. Despite insufficient walking space along streets in Tokyo, accidents have not happened frequently due to a traffic flow at low speeds.
4.3 Rail

Railway statistics in terms of the number of passengers and other factors are listed in Table 7. As indicated in Table 8, both cities have excellent city rail systems, and have served their citizens with high satisfaction. The satisfaction is based on experiences of operation and maintenance for several decades. In both cities, rail operations are kept severely on time to provide smooth and accurate transfers between various lines.

Table 7 Railway statistics for Seoul and Tokyo

<table>
<thead>
<tr>
<th></th>
<th>SEOUL</th>
<th>TOKYO</th>
</tr>
</thead>
<tbody>
<tr>
<td>Daily avg. rail passengers</td>
<td>5,600,000¹</td>
<td>8,700,000²</td>
</tr>
<tr>
<td>Rail sharing rate</td>
<td>39%¹</td>
<td>70%²</td>
</tr>
<tr>
<td>Rail distance (km)</td>
<td>331¹</td>
<td>1,054²</td>
</tr>
<tr>
<td>Rail distance / population</td>
<td>0.03¹</td>
<td>0.07²</td>
</tr>
</tbody>
</table>

2. Tokyo Metropolitan Area Transportation Planning Council, 2010

Table 8 Rail services in Seoul and Tokyo

<table>
<thead>
<tr>
<th></th>
<th>SEOUL</th>
<th>TOKYO</th>
</tr>
</thead>
<tbody>
<tr>
<td>Satisfaction</td>
<td>Mostly satisfied</td>
<td>Mostly satisfied</td>
</tr>
<tr>
<td>Comfort</td>
<td>Fairly comfortable</td>
<td>Rapid service superb</td>
</tr>
<tr>
<td>Transfer</td>
<td>No extra fares/Convenient</td>
<td>Passing through toll booths</td>
</tr>
<tr>
<td>Information services</td>
<td>Use of mobile phones widespread</td>
<td>Sufficient for rails</td>
</tr>
<tr>
<td></td>
<td>Shows the minutes to arrive</td>
<td>Shows the exact time to arrive</td>
</tr>
<tr>
<td>Fares</td>
<td>Kept low/Free to people over 65 aged</td>
<td>High fares/Subsidized by work place</td>
</tr>
<tr>
<td>Trend in demand</td>
<td>Increasing</td>
<td>Stagnant</td>
</tr>
<tr>
<td>Accessibility</td>
<td>Close to homes at stations</td>
<td>Bike transfer</td>
</tr>
<tr>
<td>Safety</td>
<td>Full screen doors at stations</td>
<td>Partial installation of screen door</td>
</tr>
</tbody>
</table>

However, at peak hours, the number of rail passengers become over capacity. To deal with this issue, Tokyo operates several rapid route services without stopping at minor stations, thus providing faster travel between major stations. In recent years, Seoul also has been started a rapid route service along congested lines.

Another difference between the two cities involves the way that transfers occur at stations. Subway passengers in Seoul do not have to pass through a tollgate at stations; however, in Tokyo, subway passengers must pay an additional fare at every transfer tollgate of different operators. In terms of queues waiting to transfer, it could be said that Seoul's system is more convenient for their passengers. Regarding arrival times, Tokyo provides accurate and exact arrival times at stations for the passengers at the platforms. Most people in Seoul, however, utilize a mobile application to obtain subway operation information.

The most remarkable difference between two cities is regarding the cost of the fares. Seoul is proud that it has one of the lowest fares in the world and is free to people over 65 years of age. Tokyo has a pension system that compensates for their high fares by means of financial subsidy from workplaces. In Seoul, there are frequent transfer passengers between rail and buses. In contrast, Tokyo operates an effective transfer system between rail and bikes. Both systems originated based on the residential demographics of the two cities. While Seoul has been structured around high-rise apartments, Tokyo has single-house dwellings; this has affected how people transfer from rail stations to their destinations.

Thanks to newly opened routes and a population increase in Seoul, the number of rail passengers also have increased. The distance people travel on rail lines in Tokyo is about 2 and a half times longer than in Seoul. The rail distance per population is 0.07 in Tokyo, which indicates that Tokyo has a greater capacity to accept more rail passengers than Seoul (0.03). Regarding the mode-sharing rate, about 70% of Tokyo's citizens have selected to travel by rail, and Seoul has about a 40% rate in the number of rail passengers. The demand...
to travel by trail in increasing year by year in Seoul and is expected to continue to grow gradually in the coming years.

4.4 Bus

Statistics for bus usage are shown in Table 9. Bus service in Seoul is so impressive, thanks to exclusive bus lanes and a superb bus information system (BIS). While Seoul bus passenger sharing rate hangs on about 30%, it seems a bit relatively low in Tokyo. Buses in Tokyo are mostly owned and operated by rail organization. Buses in Seoul are owned and operated by private companies, which are supported financially by Seoul government.

Table 9 Statistics for bus services

<table>
<thead>
<tr>
<th></th>
<th>SEOUL</th>
<th>TOKYO</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passengers</td>
<td>8,640,000$^1$</td>
<td>2,000,000$^2$</td>
</tr>
<tr>
<td>Bus sharing rate</td>
<td>27$^1$</td>
<td>3$^2$</td>
</tr>
<tr>
<td>No. of buses</td>
<td>13000$^1$</td>
<td>N/A</td>
</tr>
<tr>
<td>Buses / population</td>
<td>0.0013$^1$</td>
<td>N/A</td>
</tr>
</tbody>
</table>

2. Tokyo Metropolitan Government, Bureau of Construction, 2017

Table 10 Passenger services for buses in Seoul and Tokyo

<table>
<thead>
<tr>
<th></th>
<th>SEOUL</th>
<th>TOKYO</th>
</tr>
</thead>
<tbody>
<tr>
<td>Satisfaction</td>
<td>Fairly satisfied</td>
<td>Satisfied</td>
</tr>
<tr>
<td>No. of passengers</td>
<td>Large</td>
<td>Small</td>
</tr>
<tr>
<td>Comfort</td>
<td>Crowded at peak hours</td>
<td>Safe driving by bus drivers</td>
</tr>
<tr>
<td></td>
<td>Not efficient seating structure</td>
<td>Seats well organized</td>
</tr>
<tr>
<td>Convenience</td>
<td>Village bus to connect</td>
<td>No connected bus available</td>
</tr>
<tr>
<td>Information</td>
<td>Reliable and useful</td>
<td>Not popular</td>
</tr>
<tr>
<td>New services</td>
<td>Night bus/Double decker buses</td>
<td>N/A</td>
</tr>
<tr>
<td>Fare</td>
<td>Fares kept low</td>
<td>High fares, but commuter pass available</td>
</tr>
<tr>
<td>Ownership</td>
<td>Private</td>
<td>Rail company</td>
</tr>
<tr>
<td>Trend (Demand)</td>
<td>Increase passenger from suburban</td>
<td>Stagnant</td>
</tr>
</tbody>
</table>

The types of bus services are diverse in Seoul, providing routes to wide area around the metropolitan area. There are four common type of buses accommodating passengers along main corridor. Depending on the function, buses were divided into interregional (red), truck(blue), feeder(green), and circular(yellow) lines. By recognizing the bus lines, the city aimed to enhance mobility, accessibility, and convenience of bus services. In addition, village buses are run between transfer points and remote areas far from main corridor. This is a crucial service, and helps neighbors gain access to rail stations and main corridors conveniently.

In recent days in Seoul, a well-systemized real-time information service regarding bus operations has been shared with rails, taxis, long-distance buses, and village buses to help transfer passengers in a convenient and comfortable manner. On the other hand, bus information in Tokyo has not been as frequent for their passengers. Comparing bus fares in both cities, there is no transfer fare between bus and rail in Seoul. In Tokyo, however, bus fares are high, and include an additional transfer fare.

Bus ridership in Tokyo has remained stagnant; however, ridership in Seoul has gradually increased, especially among young passengers from cities around Seoul to the City Business District. To retain excellent bus service in Seoul, it is suggested that the city focuses on handling the gap of the number of passengers between peak and off-peak hours. In addition, demand-based bus routes might be devised more flexibly to enhance efficiency of fleet management. In Seoul, lanes exclusive for buses, known as the Bus Rapid Transit (BRT) system, has gradually expanded due to customer demand from suburbs along the main corridors.
Passengers of Seoul's BRT system has applauded its shorter travel times and comfortable seating.

4.5 Private Vehicles

As shown in Table 11, the statistics for private vehicles in both cities are similar for the indices of vehicles per population and car sharing rate. It is a usual trend in both cities to have heavy traffic due to the large number of vehicles; thus, many people avoid driving their cars to work on weekdays, leaving them in a parking lot for most of the time. At peak hours, average travel times remain below 10 km/hr., caused by recurring congestion and accidents on the roadways. Serious traffic congestion is the main reason for not driving their cars during the weekdays.

Table 11 Driving statistics for Seoul and Tokyo

<table>
<thead>
<tr>
<th></th>
<th>SEOUL</th>
<th>TOKYO</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vehicle</td>
<td>3,000,000</td>
<td>4,413,094</td>
</tr>
<tr>
<td>Vehicles / population</td>
<td>0.31</td>
<td>0.32</td>
</tr>
<tr>
<td>Car sharing rate</td>
<td>Low 20%</td>
<td>Low 20%</td>
</tr>
<tr>
<td>Average travel time</td>
<td>22 km/hour</td>
<td>N/A</td>
</tr>
</tbody>
</table>

2. Tokyo Metropolitan Police Department, 2017

As seen in Table 12, the major difference between the two cities is car ownership. While car ownership in Seoul has been growing yearly, it has remained stagnant in Tokyo. This indicates that Seoul might be able to prepare sustainably for expanding and enhancing traffic facilities around the city to accommodate an increased volume of traffic. However, improper traffic manners in Seoul hamper road safety conditions and driving/walking comfort along the corridors.

Table 12 Driving conditions for car owners

<table>
<thead>
<tr>
<th></th>
<th>SEOUL</th>
<th>TOKYO</th>
</tr>
</thead>
<tbody>
<tr>
<td>Satisfaction</td>
<td>Car ownership increased, but people not happy with recurrent traffic jams</td>
<td>Riders used to not drive on the weekdays</td>
</tr>
<tr>
<td>Comfort</td>
<td>Uncomfortable driving conditions</td>
<td>Relatively composed atmosphere</td>
</tr>
<tr>
<td>Information</td>
<td>Various mobile services available</td>
<td>Stable information system</td>
</tr>
<tr>
<td>Travel time</td>
<td>Worsening at peaks</td>
<td>Mostly congested during the weekends</td>
</tr>
</tbody>
</table>

On the other hand, traffic information is handled efficiently in both cities to help drivers search for uncongested and comfortable routes between their origins and destinations. GPS-based traffic information services are useful in providing information to drivers both prior to travel and during travel. However, regardless of having advanced information systems in both cities, recurring and non-recurring congestion and incidents inevitably have occurred at the CBDs and on major streets. Resolving habitual issues must be the most critical policy addressed for both cities.

4.6 Traffic Manners

As seen in Table 13, there are remarkable differences regarding traffic manners between the two cities. While most vehicles in Tokyo have followed the traffic laws and rules, most vehicles in Seoul violate the traffic laws. These behaviors result in big difference in reality. In general, a violation usually results in a total disorder in the traffic flow. Without severe enforcement for traffic make worse in driving and walking along roads. In fact, usually it is called a 'traffic hell' in Seoul. In 2017, the region around Seoul had about 2.5 million violations; these are just the cases that were enforced by the police. The number probably is much larger than 2.5 million.
In Seoul, many types of traffic violations that disturb the normal movement of traffic. Speeding and traffic signal violations comprise more than 50% of all traffic violations. Out of the total types of vehicles on the roads, passenger vehicles are involved in 70% of the traffic violations. About 90% of the violations by all vehicles have been identified and enforced by using an unmanned device, such as CCTV which were monitored and ticketed for a traffic violation. Even so, drunk driving and reckless driving correlate closely to traffic fatalities resulting from threatening other vehicles. However, these types of violations are difficult to enforce without disturbing the traffic flow; thus, these disruptions quickly become widespread throughout the metropolitan area.

The penalty levels in Seoul are too low for people to be willing to stop violating the law; however, the penalty levels in Tokyo are high enough to encourage drivers to obey the laws in a proper manner. The difference of penalty between two cities is about five to ten times. Additionally, traffic enforcement in Tokyo is severe, and is enforced consistently anywhere around the metropolitan area. In contrast, enforcement policies in Seoul have not been severe enough to prohibit vehicles from violations; In Seoul, they have been often affected by external factors, such as politics and administrative matters, and so they remain unsustainable to implement.

Walking in Seoul is very dangerous due to reckless drivers who park their cars along sidewalks and other walkways. Moreover, traffic manners in Seoul are very bad and getting worse year by year. On the other hand, Tokyo has improved issues regarding traffic manners by consistent effort for decades, including enforcing harsh penalties. Thanks to long-standing efforts by the Tokyo government and its people, traffic rules are obeyed well enough that most violations are voided when driving and walking. Driving in Tokyo has become comfortable and convenient due to the good nice traffic manners around the Tokyo metropolitan area.

4.7 Parking

Usually, parking is a meaningful way to determine the level of transportation in a city or region for both residents and tourists. Most of the time, vehicles usually stay in one spot, such as a parking lot, rather than moving around, which would indicate that the management of parking should be a priority over the movement of vehicles throughout the city. Tables 14 and 15 provide survey results and statistics that compare parking in Seoul and Tokyo.

Table 13 Comparison of traffic manners in Seoul and Tokyo

<table>
<thead>
<tr>
<th>Types of violation</th>
<th>SEUL</th>
<th>TOKYO</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stop Signs</td>
<td>Not following stop-sign directions</td>
<td>Fairly good compliance</td>
</tr>
<tr>
<td>Speeding</td>
<td>Prevalent</td>
<td>Not frequent</td>
</tr>
<tr>
<td>Drunk Driving</td>
<td>Escort driver services prevalent</td>
<td>Not frequent</td>
</tr>
<tr>
<td>Penalties</td>
<td>Remain at a low level</td>
<td>Fairly high level of penalties</td>
</tr>
<tr>
<td>Treatment</td>
<td>Weak &amp; irregular</td>
<td>Severe</td>
</tr>
<tr>
<td>Pedestrians</td>
<td>Risky</td>
<td>Safe</td>
</tr>
</tbody>
</table>

Table 14 General comparison of parking in Seoul and Tokyo

<table>
<thead>
<tr>
<th></th>
<th>SEUL</th>
<th>TOKYO</th>
</tr>
</thead>
<tbody>
<tr>
<td>Satisfaction</td>
<td>Not mostly</td>
<td>Mostly</td>
</tr>
<tr>
<td>Parking lot</td>
<td>Shortage</td>
<td>Sufficient</td>
</tr>
<tr>
<td>Penalty</td>
<td>Fairly low</td>
<td>High enough that people do not violate the traffic laws</td>
</tr>
<tr>
<td>Treatment</td>
<td>No rational plan</td>
<td>Sustainable and severe</td>
</tr>
</tbody>
</table>
Encourage parking schemes. Those inconsistent parking strategies have resulted in a shortage of parking space, thus cause serious traffic jams around the areas. Vehicles illegally parked have led to reduced space for driving in traffic lanes, thus cause serious traffic jams around the areas.

One remarkable difference between Tokyo and Seoul is regard to their parking policies and management. Tokyo has actively implemented a 'parking lot requisition' policy since 1980s. Thanks to their unique parking policy, it is rare to see vehicles parked illegally along streets, especially around residential areas; most vehicles have been legally parked inside the property of the home. Additionally, plenty of parking lots can be found easily in neighborhoods, which reduces the temptation to park illegally. With vehicles parked according to traffic regulations, moving vehicles can be driven around the neighborhood without obstruction from vehicles illegally parked on roadways. Tokyo has more than 200% parking supply capacity while Seoul shows about 130%.

Unlike Tokyo, Seoul has experienced many cases of illegal parking around the city. To enforce the laws for vehicles parked illegally along streets and walkaways, many CCTVs have been installed and operate in many locations. However, this has not been found effective to deter illegal parking because drivers are aware of where the CCTVs are located. Furthermore, emergency vehicles often cannot reach the right place to implement their jobs due to cars parked illegally along streets. Vehicles illegally parked have led to reduced space for driving in traffic lanes, thus cause serious traffic jams around the areas.

Regarding penalties for illegal parking, Tokyo has much more severe penalties than does Seoul. Besides, parking policies in Seoul has been revised frequently without expanding parking spaces and enhancing parking schemes. Those inconsistent parking strategies have resulted in a shortage of parking space, thus encourage habitual behaviors regarding illegal parking behaviors.

5. ENHANCEMENT STRATEGIES BY VARIOUS CATEGORIES

5.1 Enhancement strategies

Table 15 Penalties for illegal parking in Seoul and Tokyo

<table>
<thead>
<tr>
<th></th>
<th>SEOUL</th>
<th>TOKYO</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parking supply capacity</td>
<td>130%</td>
<td>More than 200%</td>
</tr>
<tr>
<td>Maximum penalty fee for illegal parking</td>
<td>$70</td>
<td>$200</td>
</tr>
<tr>
<td>Penalty points</td>
<td>0</td>
<td>3</td>
</tr>
</tbody>
</table>

2. Tokyo Metropolitan Police Department, 2017

Table 17 Action plan by categorized subjects

<table>
<thead>
<tr>
<th>Category</th>
<th>Subjects</th>
<th>Seoul</th>
<th>Tokyo</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transit</td>
<td>Policy</td>
<td>Needs better bus-subsidy policy</td>
<td>Adjust the transit fare system</td>
</tr>
<tr>
<td></td>
<td>Facility</td>
<td>Develop new light-rail routes</td>
<td>Provide more transfer information</td>
</tr>
<tr>
<td></td>
<td>Operation</td>
<td>Expand rapid train service</td>
<td>Avoid delays at peak hours</td>
</tr>
<tr>
<td>Private</td>
<td>Travel times</td>
<td>Activate pricing strategies</td>
<td>Manage recurrent traffic jam properly</td>
</tr>
<tr>
<td>Vehicles</td>
<td>Convenience</td>
<td>Enforce strictly for violation</td>
<td>Rejuvenate aging infrastructure</td>
</tr>
<tr>
<td>Safety</td>
<td>Accidents</td>
<td>Build more safety facilities</td>
<td>Install unmanned facilities to enforce</td>
</tr>
<tr>
<td></td>
<td>Circumstance</td>
<td>Enforce strictly on walkaways</td>
<td>Educate reckless bike riders</td>
</tr>
<tr>
<td></td>
<td>Policy</td>
<td>High penalty to reckless drivers</td>
<td>Invest more on safety facilities</td>
</tr>
<tr>
<td>Green</td>
<td>Policy</td>
<td>Limit diesel vehicles</td>
<td>Do pricing schemes</td>
</tr>
<tr>
<td></td>
<td>Nonmotorized</td>
<td>Build safe route for bikes</td>
<td>Expand exclusive bike roads</td>
</tr>
<tr>
<td></td>
<td>Exclusive</td>
<td>Build more pedestrian malls</td>
<td>Build time-flexible malls</td>
</tr>
<tr>
<td>New</td>
<td>Modes</td>
<td>Build a street tram</td>
<td>Activate more recharging stations</td>
</tr>
<tr>
<td></td>
<td>Autonomous</td>
<td>Invest in R/D</td>
<td>Invest in R/D</td>
</tr>
<tr>
<td></td>
<td>Sharing</td>
<td>Allow private cars to share</td>
<td>Allow private cars to share</td>
</tr>
<tr>
<td>Traffic</td>
<td>Enforcement</td>
<td>Establish stricter policies</td>
<td>Keep current enforcement schemes</td>
</tr>
<tr>
<td>manner</td>
<td>penalty</td>
<td>Increase penalty level</td>
<td>Keep current strategies</td>
</tr>
<tr>
<td></td>
<td>Education</td>
<td>Initiate strong lessons in school</td>
<td>Upgrade the level of education</td>
</tr>
<tr>
<td>Parking</td>
<td>Supply</td>
<td>Provide more parking spaces</td>
<td>Keep the well-balanced scheme</td>
</tr>
<tr>
<td></td>
<td>Facilities</td>
<td>Allow street parking</td>
<td>Improve accessibility to parking lots</td>
</tr>
<tr>
<td></td>
<td>Manner</td>
<td>Strengthen parking enforcement</td>
<td>Keep effective strategies in place</td>
</tr>
</tbody>
</table>
5.2 Answers for questions

<table>
<thead>
<tr>
<th>Table 18 Feedback from stakeholders from both cities</th>
</tr>
</thead>
<tbody>
<tr>
<td>Are the citizens satisfied with the transport system?</td>
</tr>
<tr>
<td>Yes, citizens in both cities are satisfied in general with the transportation services.</td>
</tr>
<tr>
<td>Are the existing transport systems suitable to accommodate demands?</td>
</tr>
<tr>
<td>Yes, transit in cities are well-managed to provide suitable services. However, Seoul should be more reactive to a trend of an increasing number of vehicles.</td>
</tr>
<tr>
<td>Are the transport modes efficiently connected and collaborated?</td>
</tr>
<tr>
<td>Yes, collaboration between modes has been enhanced; however, more work is required.</td>
</tr>
<tr>
<td>Which strategies could be possible to provide good solutions to traffic congestion issues?</td>
</tr>
<tr>
<td>TDM (Transportation Demand Management) would be proper to handle it, especially adapting intelligent transportation system (ITS) and related approaches.</td>
</tr>
<tr>
<td>What are the policies to reduce a high number of serious traffic accidents?</td>
</tr>
<tr>
<td>Adjusting the penalty level, depending on the type of traffic manners, and strengthening traffic enforcement, using a sustainable scheme.</td>
</tr>
</tbody>
</table>

6. CONCLUSION

The following lessons can be drawn from comparing the two cities of Seoul and Tokyo in terms of performance of their transportation systems.

Both cities are similar in size but have different traffic situations. Seoul and Tokyo are similar in terms of area (land) and population; however, traffic conditions are somewhat different in aspects of various categories. Each city has its own various merits and demerits. To keep traffic moving smoothly and uncongested along streets, more efficient and sustainable policies are required. Furthermore, results from these policies could be benchmarked for future comparisons.

Seoul might benefit from a best practices study of Tokyo for improving traffic manners. Illegal traffic behaviors frequently disturb normal traffic conditions in Seoul, resulting in unnecessary and unexpected traffic jams and traffic accidents. Traffic conditions in Seoul could be enhanced largely by a return to normal traffic manners; in fact, Tokyo has proven that city traffic can operate due to people exhibiting good traffic manners.

Tokyo could learn from Seoul how to control traffic efficiently for Bus Rapid Transit and Bus Information System (BIS) services. In Seoul, BRT is a sort of High Occupancy Vehicle (HOV) system and was initiated in 1990s. Seoul has decided to make using buses a priority over private vehicles with more than two people. Over the past couple decades, this system has operated successfully to reduce congestion and travel time along suburban and urban corridors in Seoul. Implementing a BRT system in Tokyo might be useful to control traffic, encouraging more people to use buses. In addition, an advanced BIS would be a valuable solution for transit users around the Tokyo metropolitan area.

Transit ridership in both cities reached 70%, which might be a maximum threshold for mega-cities. Usually balancing between transit and private vehicles would be a crucial factor in controlling traffic effectively around the entire metropolitan areas of mega-cities. These two cities showed 70% occupancy in using transit, meaning that less than 30% used other transportation modes, such as pedestrians, bikes, taxis, and private vehicles. If transit had exceeded over 70% in a ride-sharing rate, many private vehicles would have stayed in a parking lot for a longer period. It is recommended that policy makers focus on improving the existing transit services in both cities rather than build new infrastructures, such as new routes and more trains.

Operation and maintenance must be more important than planning for cities having a population of more than 10 million. The two cities investigated in this study had a significant amount of traffic infrastructure and systems. It would require substantial financial expenditure and efforts to build new infrastructures and systems, which is not economical. Utilizing existing facilities more efficiently would be much better in terms of a cost and benefit analysis; therefore, improving operations and concentrating on maintenance projects...
would be more important right now. Currently, there are many traffic system management skills and transportation-demand management schemes available to upgrade the efficiency of systems already in place. It is recommended that both cities look for customized solutions to improve their existing facilities, perhaps drawing 'best practices' from each other.

**Urban structural conditions affect building and managing transportation facilities.** About residences, Tokyo has a sprawl shape, with the greatest density in the CBD and gradually lessens out to suburban areas. Therefore, a densely webbed transit network is required to accommodate transportation demands separated far from stations. On the other hand, Seoul has multiple peaks of density in the CBD as well as in other residential areas. Apartments in high-rise developments in Seoul have a different city structure compared to Tokyo. This difference in the two cities result in dissimilar transportation-demand generation patterns and transfer preference; as a result, they have different transit systems and road networks.

It is recommended that the merits of each city be benchmarked by means of frequent meetings and regular exchanges by personnel. Sharing the 'best practices' of each city with each other would help enhance transportation services and system performance of both cities over a short period of time. If the two cities exchange ideas and experiences of their personnel effectively, they might be able to reduce budget expenditures and reduce the amount of new facilities built. Difficulties in operation and maintenance also could be shared to decrease the erratic implementation of policies.

The next study will focus on developing evaluation standards based on analyzing and categorizing the characteristics of each city. These standards could have tested in sample cities. To conduct this study, an expert group should be organized that includes expertise from such fields as transportation, urban planning, statistics, administrative, and industries.

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ABSTRACT:
Road Transportation Ministries and asset managers around the world usually require annual inspections of their roads and infrastructures to plan maintenance operations. Using road surface defects, likely measured using 3D laser sensors, as input data to Pavement Management System (PMS) software they can determine if the road needs to be rehabilitated and resurfaced. In this situation, a high precision survey of the surface will usually be required and used as input to 3D CAD road design software that can then be output to control 3D pavers and millers using laser tracking total stations.

In this article we will present a new approach to create the road surface model by reusing the 3D road surface condition data and that avoids expensive manual road surface surveys. By combining high resolution and high accuracy data acquired by a LCMS system (Laser Crack Measurement System – Pavemetrics) with highly accurate GNSS-INS system (Applanix POS-LV - Trimble) we can measure both road surface conditions and generate a survey grade accuracy terrain map of any road surface. We will describe how the information provided by both systems is used to generate the road surface models and will compare the accuracy and repeatability of multiple runs of these models with surveyed control points.

Results will show that by using this method we can generate much higher resolution survey grade road surface models for resurfacing applications using 3D paving and milling equipment with significant productivity improvements, material optimization, lower survey cost, decreased traffic interruption and improved safety.
Using network level 3D road surface data for project level 3D paving and milling

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INTRODUCTION

Road Transportation Ministries and asset managers around the world usually require annual inspections of their roads and infrastructures in order to plan maintenance operations. Road surface defects (texture, cracks, rutting, IRI – smoothness) are important data that need to be measured and serve as input data to PMS (Pavement Management Systems) software. These defects are likely measured using 3D laser sensors that acquire the shape of the road surface in order to evaluate its condition (Laurent et al. 2008) (Li et al. 2011) (Wang 2011) (Wix & Leschinski 2013) (Laurent et al 2017). Once it is determined that the road condition has degraded to the point that it needs to be rehabilitated and resurfaced then a high precision survey of its surface is usually required. This survey is typically used by the engineers as an input to 3D CAD road design software that can then be output to control 3D pavers and millers using laser tracking total stations.

Until now, the only way to capture a highly accurate representation of the road surface was to mandate surveyors with laser total stations to sample each lane over the entire length of the project. This action requires a lot of human resources and necessitates the closure of lanes to traffic.

The approach that we propose is to combine data from different sensors (3D laser profilers, inertial system, GNSS and DMI) to produce a real 3D representation of the road surface that is as accurate as the one produced by a surveyor but with a higher density of data.

1. SYSTEM CONFIGURATION

The 3D mapping solution proposed is based on data provided by 2 laser profilers that acquire 4000 points 3D transverse profiles of a road lane up to 4m wide. These sensors can operate at profile rates as high as 28,000Hz allowing the acquisition of a transverse profile at 1mm intervals at speeds up to 100km/h. These 3D point clouds are usually processed to extract road surface distresses such as cracks, ruts, pot holes and even evaluate aggregate loss and surface texture.

To acquire real 3D surface maps of the road the 3D data from the laser profilers must be corrected to compensate for vehicle and suspension motion, driver wander and vibrations. To achieve this a Distance Measuring Instrument (DMI) and sensors capable to provide inertial information about the attitude of the vehicle/sensors installation and its location must be added to the system. To achieve this a Global Navigation Satellite System (GNSS) and an Inertial Measurement Unit (IMU) or an Inertial Navigation System (INS) that integrates both functions must be added to the system.
2. SYSTEM CALIBRATION

System calibration is required to map the different physical locations and coordinate systems of the systems components (GPS, IMU, Laser Sensor, DMI) on the survey vehicle into a single final reference system (GPS). The purpose of the calibration is to establish the digital model of the position of 3D sensors on the vehicle versus the position of the other elements (GPS, IMU, DMI).

The calibration process consists in four steps.

2.1 Physical measures of the different level arms between each system sensors.
2.2 Scan of a reference

That step of the calibration process establishes the necessary parameters to solve the ambiguity between the left and right LCMS sensors. The overlap zone between the sensors over the reference object will be used as a reference surface and the adjustments done on the different metrics (orientation, angles, height, etc.) of the LCMS sensors to produce a perfect surface will be used in the final calibration solution.

2.3 “Stop” and “Go” calibration

The “Stop And Go” part of the calibration will measure the acceleration in the 3 axes (X, Y and Z) of the IMU when the vehicle is stationary and when it is accelerating. This information is used to determine the orientation of the sensors related to the gravitational force.

2.4 Calibration Runs

3 specific survey type calibration runs are required passing over the reference object and are done to fine tune the calibration parameters (gyro and accelerometer bias) to ensure a perfect match between sensors and runs (Stitching). This step will determine the necessary parameter adjustments required to compensate for the drift over time of the IMUs.
Figure 5. Run 1: Loop

Figure 6. Run 2: Back Thru

Figure 7. Run 3: Right angle

3. PROCESSING

3.1 GNSS/INS systems

To obtain the best accuracy possible, it is highly recommended to post-process the GNSS/INS data using a local base station with RTK corrections. The GNSS/INS data processing is usually done by the software provided by the INS/GNSS manufacturer. The output of the GNSS/INS post-processing will be fed into the Terrain Mapping software and will replace the real-time navigation solution recorded during the survey even if there is no post-process data available, it is always possible to produce a 3D surface using the real-time data recorded in the files during the data acquisition however accuracy and repeatability will be affected.
3.2 Terrain Mapping

The terrain mapping processing consist on the assignation of an accurate location (Lat/Long, UTM) to each pixels of every profile contained in the LCMS survey.

3.3 Necessary steps

3.3.1 Navigation solution (Post-processed or not)

The navigation solution provides all the information on the attitude and movements of the vehicle. This information can be applied through the calibration parameters to the LCMS data. At this step, the resulting surface is as precise as the navigation solution. The accuracy of the surface can be improved by the usage of alignment and control points as described in the following steps.

![Navigation solution](image1)

Figure 8. Navigation solution

![Uncorrected 3D mesh data of three different lane scans](image2)

Figure 9. Uncorrected 3D mesh data of three different lane scans (the overlapping areas do not match up well).
3.3.2 Tie points

To perfectly stitch together many lanes coming from many runs or surveys, tie points need to be created in the overlap zone between these lanes. Tie points are common features that are present in 2 different runs. These points are used to stitch runs all together and to produce a unique surface or point cloud.

These points can be found automatically by the detection software using 3D correlation of the overlap zone.

3.3.3 Applying surveyed reference Alignment points

The alignment points are used to attach the surface to a highly accurate reference point. These points are normally surveyed using a robotic total station and are separated from each other of about 300 to 1000 meters on road surface or shoulder. These points are then imported in the processing software to be added to the positioning solution. After that step, the entire point cloud is as accurate as a survey grade survey which means 3 to 5 mm.
4. RESULTS/VALIDATION

In order to establish the accuracy of the system, a reference site was build and more than 500 reference points.
These 500 points were surveyed 3 times using a robotic total station and a laser level. The accuracy of the ground truth which was established to be between 2 and 5 mm.

The Laser Digital Terrain Mapping (LDTM) system (LCMS + POS LV) was then evaluated on the basis of the ground truth and 12 runs were done to evaluate the repeatability and the accuracy of the system.

Two alignment points distances were tested and evaluated, 300 and 850 meters.

The results show that with a distance of 300 meters between the alignment points (control) that the accuracy and the repeatability of the LDTM system were averaging between 2 and 5 mm. For the 825 meters scenario, the results showed an overall accuracy of 5 to 9 mm and for the repeatability 4 to 6 mm.
5. APPLICATION

The Terrain Mapping (LDTM) system can export the surface model data in LAS format that can be read by any 3D CAD software. Before exporting the software will scale the resolution of the data to lower and more manageable point densities. Very nice detailed maps can be generated at resolutions of 100 x 100mm. Points are decimated to fixed resolutions (x,y) and the vertical (z) position is filtered to avoid reporting rocks and cracks on the road surface.

![Diagram](http://construction.trimble.com/)

Figure 17. LDTM Solution for road rehabilitation

These high-resolution 3D surface models can be used by engineers to design better roads (KilPelainen et al. 2011) (Gräfe 2005) (Gräfe 2008) compared to the traditional survey method that only generates 3 points across the road every 20m or lidar based mobile mapping solutions that are only accurate to a few centimeters.
This increased resolution allows to better optimize the quantity of material that needs to be carried in and out of the construction site. The use of LDTM models and automated laser controlled milling machines that adjust the height of the cutting heads can be used to correct the roads longitudinal profile (compared to fixed milling depth and 3D controlled Pavers can then be used to create variable thickness new asphalt layers also resulting in a smoother road profiles that will last longer considering the reduced axial dynamic loads resulting from the heavy vehicles.

CONCLUSION

A method was described that combines data from different sensors (3D laser profilers, inertial system, GNSS and DMI) to produce at traffic speed a real 3D representation of the scanned surface. Multiple lanes of a road surface were scanned one at a time and merged automatically together using 3D features detected in common overlapping areas between the scans of adjacent lanes. The resultant road surface map was adjusted to ground control points and the accuracy and repeatability of the overall surface model was evaluated to be as good as those that can be produced by a surveyor with a laser total station.

Using this methodology, it was shown that it is possible to generate much higher resolution survey grade road surface models that can be used for resurfacing applications using 3D paving and milling equipment from the original 3D data that was used to evaluate the actual condition of the road surface itself. This process results in significant productivity improvements from lower survey costs, decreased traffic interruptions and improved safety of surveyors while improving the quality and resolution of the road surface models.

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Wix, R & Leschinski, (2013). 3D Technology for managing pavements., Institute of Public Works Engineering Australia conference, Darwin, Australia
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**KEYWORDS:**
Pavement Management, Performance Targets, MAP-21, FAST

**ABSTRACT:**
This presentation demonstrates an innovative approach for network level pavement management programs development for agencies. The goal is to minimize budget demands while maintaining the performance targets or increasing the return on investment (ROI) for network level pavement management. This methodology is unique as it considers the concepts and rulings for MAP-21 and FAST Acts performance measure target setting for pavements. It aims at helping agencies to carry out intelligent and practical annual and multiple years network level pavement work programming. It shows how agencies can evaluate and define their performance measure targets for pavements based on MAP-21 and FAST Acts.

This presentation reports the results of a case study that shows how agencies can define performance targets based on the condition of the network. Establishing several candidate scenarios including different intervention strategies to accomplish agency network performance goals.

It shows utilizing multi-constraint optimization analyses to evaluate the ROI of each network level scenario while satisfying applicable constraints. The case study demonstrates how proactive approaches with more focus on preservation can help reduce the costs of meeting performance targets while better sustaining long-term network condition.
Establishing Practical Network Level Pavement Management Program Using Smart Performance Targets and Optimization Analysis

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Senior Principal Consultant
AgileAssets Inc, Austin, TX, USA
1.0 INTRODUCTION

Road management agencies worldwide deal with the question of performance target setting for pavements and increasing the return on investment (ROI) on maintenance and rehabilitation budgets. This study demonstrates an innovative and practical approach for network level pavement management program development for roadway agencies. This approach minimizes budget demands while maintaining the pavement performance targets within the framework of network level pavement management. It is critical for USA State DOTs and MPOs to consider the FHWA MAP-21 and FAST programs requirements regarding performance management for pavements.

2.0 TRANSPORTATION PERFORMANCE MANAGEMENT (TPM)

FHWA defines Transportation Performance Management (TPM) as a strategic approach that uses system information to make investment and policy decisions to achieve national performance goals. Transportation Performance Management provides key information to decision makers allowing them to understand the consequences of investment decisions across transportation assets. Roadway infrastructure management agencies should ensure that measures and targets are developed in cooperative partnerships and should be based on data and objective information. Figure below shows key building blocks in performance management program implementation.

2.1 Performance Based Planning and Programming

Figure below shows typical steps for agencies to iteratively analyze and consequently improve their asset management plans and programs. This process requires establishing initial performance goals and quantifiable performance measures for road assets. Then robust asset/pavement management tools are used to create alternative strategies to meet agency performance goals. Finally, optimal plans are prepared and implemented on the network each year. It is critical to monitor and track asset management programs progress and evaluate the results using quantifiable performance parameters. Results and lessons learned are used to improve targets forecast and goals for the next cycle. The process continues cyclically, and agencies will achieve more reliable programs and forecasts.

2.2 Pavement Condition Minimum Thresholds

Following pavement condition/performance measures are established under FHWA MAP21 program:
- Ride Quality – International Roughness Index (IRI)
- % Cracking
- Rutting (Asphalt Pavement) /Faulting (PCC Pavement)
Table below lists Pavement condition metrics thresholds as per MAP21 FAST Act (490.311). These include three rating levels (Good, fair, Poor) for each pavement performance measure.

<table>
<thead>
<tr>
<th>Metric</th>
<th>Good</th>
<th>Fair</th>
<th>Poor</th>
</tr>
</thead>
<tbody>
<tr>
<td>IRI (inches/mile)</td>
<td>&lt;95</td>
<td>95-170</td>
<td>&gt;170</td>
</tr>
<tr>
<td>Cracking (%)</td>
<td>&lt;5</td>
<td>Asphalt: 5-20</td>
<td>Asphalt: &gt;20</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CRCP: 5-10</td>
<td>CRCP: &gt;10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Jointed: 5-15</td>
<td>Jointed: &gt;15</td>
</tr>
<tr>
<td>Rutting (inches)</td>
<td>&lt;0.20</td>
<td>0.20-0.40</td>
<td>&gt;0.40</td>
</tr>
<tr>
<td>Faulting (inches)</td>
<td>&lt;0.10</td>
<td>0.10-0.15</td>
<td>&gt;0.15</td>
</tr>
</tbody>
</table>

2.3 Pavement Condition Data Collection Requirements

State DOTs are required to establish performance targets representing the full extent of mainline highways National Highway System (NHS) network including through travel lanes only. This excludes ramps, shoulders, turn lanes, crossovers, etc. Hence, main applicable roadway Network is NHS. There is 5% allowable limit for missing/invalid/unresolved network data while collecting performance data over the network.

Highway performance Management System (HPMS) field manual for pavement data collection protocols are used for key performance indicators field data collection. State DOTs are required to report condition for each tenth of a mile pavement section.

2.4 Pavement Measures Rating Calculation

Matrix below shows the assignment of overall pavement section rating, Good/Fair/Poor, based on the combination of individual IRI/Cracking/Rutting ratings for Asphalt concrete pavements:

- Overall Rating is “Good”, when all 3 IRI/Cracking/Rutting are Good.
- Overall Rating is “Poor”, when at least 2 measures IRI/Cracking/Rutting are Poor.
- Overall Rating is “Fair”, all other combinations.

<table>
<thead>
<tr>
<th>Overall Condition Rating</th>
<th>Asphalt Concrete</th>
<th>PCC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>IRI, Cracking, Rut</td>
<td>IRI, Faulting</td>
</tr>
<tr>
<td>Good</td>
<td>All 3 Measures “Good”</td>
<td>All 2 Measures “Good”</td>
</tr>
<tr>
<td>Poor</td>
<td>2 or 3 Measures “Poor”</td>
<td>All 2 Measures “Poor”</td>
</tr>
<tr>
<td>Fair</td>
<td>All Other Combinations</td>
<td>All Other Combinations</td>
</tr>
</tbody>
</table>

Example for an asphalt surfaced interstate section rating example is shown below. First, three performance measures IRI, Cracking, and Rutting numerical values are converted into individual “Good/Fair/Poor” ratings. These individual ratings are used to derive an overall section rating based on matrix above. The example section overall rating was established as “Fair” based on the abovementioned matrix.
2.5 Target Setting, Minimum Condition Threshold & Penalty

Table below lists overall Pavement condition measure as per MAP21 FAST Act. Roadway network management agencies mainly USA State DOTs are required to set pavement asset targets for %Good and %Poor conditions.

<table>
<thead>
<tr>
<th>Pavement Condition Performance Measures</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>(NHS System: Interstate &amp; Non-Interstate NHS)</td>
<td></td>
</tr>
<tr>
<td>% Pavements in <strong>Good</strong> Condition</td>
<td></td>
</tr>
<tr>
<td>% Pavements in <strong>Poor</strong> Condition</td>
<td></td>
</tr>
</tbody>
</table>

FHWA program has established minimum target threshold for overall Pavement condition measure – which is set as:
- “No more than 5% Poor condition.”

5% minimum overall pavement performance measure for the agency pavement network is mandated.

The Penalty proposition established for not meeting the minimum condition threshold is that:
- “State must Transfer a specified % from Surface Transportation Program (STP) funds to address Interstate conditions”.

Hence State will have to cut some other work programs to meet the NHS minimum condition goals.

2.6 Performance Reporting

FHWA require multiple reports over a four-year reporting performance period.
- Baseline Performance Report: Initial report due: October 2018
- Full Performance Period Progress Report: 2-year and 4-year targets reports.

3.0 ESTABLISHING PERFORMANCE TAGRETS & WORK PROGRAMS USING PMS

Pavement management systems (PMS) are used by highway agencies to establish best possible network level maintenance and rehabilitation work programs for the road network. Producing a network level work program that applies the right treatment at the right time on network pavement sections is central to the success of a pavement management program. PMS help in establishing the funding levels required to meet agency desired pavement performance or level of service goals. AgileAssets Pavement AnalystTM optimization analyses handles this problem at the network level and generate optimization-based network level work program which provides overall best possible returns (1, 2).

3.1 Pavement Network Optimization Analysis

AgileAssets Pavement AnalystTM aims to determine the most efficient work program that yields maximum benefit for the public funds. However, the work programs are based on several underlying input variables, models and equations, for example: treatments, unit costs, decision trees, prediction models and other inputs used by the agency. It is very important that agencies develop reliable estimates for the critical inputs to the systems. Flow diagram below shows the work flow in using the PMS in generating the multi-constraint optimization-based work program.
3.2 Performance vs Treatment Categories

Newly built or rehabilitated pavements typically exhibit higher levels of performance measures values. However, pavement performance gradually decreases over time under traffic loading and environmental factors. Pavement performance models for different performance measures such as IRI, Cracking, Rutting should be configured in the PMS to predict future conditions over the analysis period.

Roadway agencies goal is to develop maintenance and rehabilitation work programs aiming to sustain target performance standards at optimal costs. Agencies use decision trees in the process to assign the treatments to pavement sections based on agency’s used decision trees logic. Pavement strategies can include a variety of approaches including more frequent lower costs preventive treatments compared to performing higher cost rehabilitation treatments when condition reaches to certain defined thresholds. Figures below illustrate graphically the general trend in performance values overtime under the two general scenarios mentioned here. Figures below also list some typical treatments and decision tree example.

3.3 Using Pavement Analyst™ to Evaluate Performance Targets and Budgets

AgileAssets Pavement Analyst™ system provide robust capabilities to conduct variety of network optimization scenarios. Two most common network analysis scenarios are:

- Fixed annual budgets constraint and objective to maximize network
- Calculate minimum budget levels to meet annual performance/condition goals.

AgileAssets “Multi-Constraint” network Analysis allows handling of multi-constraint analysis problems. For example, different budget levels for administrative units (districts), or by different performance thresholds for different pavement functional classes. Moreover, attaining the maximum network condition possible for the maintenance fund available while restricting the number of deficient miles to a certain threshold. Pavement Analyst™ output is “Optimal” set of projects that has max/min objective while meeting all constraints. Below are some typical “What if” Scenarios which can be analyzed by using Pavement Analyst™ Optimization Analysis.

1. Performance Target X, Statewide NHS, Determine Cost?
   - Objective – Minimum Cost
   - Constraint - Poor Less than X%

2. Performance Target X & Y, Statewide NHS, Determine Cost?
   - Objective – Minimum Cost
   - Constraint - Poor < than X% (5%)
   - Constraint - Good > than Y

3. Performance Target X & Y, Statewide NHS, Determine Cost?
   - Objective – Maximize Performance (IRI)
   - Constraint - Poor < than X% (5%),
   - Constraint – Budget Not to Exceed $M

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3.4 Network Optimization Analysis Scenarios

In this study we evaluated 5 network pavement network analysis scenarios for a sample road network. All scenarios are conducted using 10 years Analysis Period. Each scenario had a different set of objective and constraints to model the FHWA performance threshold targets and ranges in the scenarios.

<table>
<thead>
<tr>
<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>Minimize Cost Poor &lt; 5%, Good &gt; 20%</td>
<td>Minimize Cost Poor &lt; 5%, Good &gt; 28%</td>
<td>Maximize Perf– IRI Poor &lt; 5%, Budget - $100 Mil/Yr</td>
<td>Maximize Perf – IRI Poor &lt; 5%, Budget - $75 Mil/Yr</td>
<td></td>
</tr>
</tbody>
</table>

1. Scenario 1 is to find the work program meeting the minimum condition goal, it will find work program based on minimizing total cost (objective) each year while meeting following constraint “Poor < 5%”.

2. Scenario 2 is multi-constraint, it will find work program based on minimizing total cost (objective) each year while meeting following two constraints, “Poor < 5%” and “Good > 20%” each year.

3. Scenario 3 is multi-constraint, it will find work program based on minimizing total cost (objective) each year while meeting following two constraints, “Poor < 5%” and “Good > 28%” each year.

4. Scenario 4 is multi-constraint, it will find work program based on maximizing performance measure (IRI) (objective) each year while meeting following two constraints, “Poor < 5%” and “Budget 100 mil/year”.

5. Scenario 5 is multi-constraint, it will find work program based on maximizing performance measure (IRI) (objective) each year while meeting following two constraints, “Poor < 5%” and “Budget 75 mil/year”.

3.5 Summary of Scenario Analyses Results

All scenarios listed above generate a list of projects distributed over 10 years period. Each project has a treatment and estimated cost. Each segment has calculated performance measures for IRI, Cracking, and Rutting for each year. All these variety of statistics have been summarized and reported in the charts below.

Seven charts are shown below respectively:

- Distribution of %Poor network each year for each scenario,
- Distribution of %Good network each year for each scenario,
- Distribution of Annual Budgets ($Mil) each year for each scenario,
- Cumulative distribution of Annual Budgets ($Mil) each year for each scenario,
- Distribution of IRI value each year for each scenario,
- Distribution of performance measure (Good/Fair/Poor) for each scenario.
- Distribution of performance measure (Good/Fair/Poor) at year 10 for each scenario.

Chart a: “%Poor” Pavement Measure Trend for Each Scenario
Chart b: “Good” Pavilion Measure Trend for Each Scenario

Chart c: Annual Budgets ($Mil) for Analysis Scenarios

Chart d: Cumulative Budgets Estimates ($Mil) for Analysis Scenarios

Chart e: Distribution of IRI value each year for each scenario
Following observations have been made reviewing the summary results charts below.

- All scenarios show that overall %Poor is less than 5%. This is achieved since each scenario was setup with the constraint setup for %Poor network to stay below 5%.

- Scenario 1 has only constraint for min %Poor; hence analysis didn’t try to fix more once %Poor constraint is met. Therefore, %Good drops from 16.6% at the start of the analysis to 2.7% at the end of 10 years. Scenario 1 spent $57 Mil in year 1 to make the network %Poor below 5%. After that the budget demand was reduced significantly for future years to sustain that condition threshold. Total 10-year budget of $202 Mil is estimated for 10 years.

- Scenarios 2 & 3 has constraint for performance target %Good 20% and 28% respectively in addition to %Poor constraint of 5%. These scenarios produce the work plan meeting these two constraints at the lowest budgets annually. This obviously produces varying budget levels for each year. For Scenario 2, annual budget demand range is from $59 mil to $92 mil during 10-year period. For Scenario 3, annual budget demand range is from $77 mil to $125 mil during 10-year period. Scenario 3 has 28% Good target compared to 20% for Scenario 2, this is the obvious reason for the higher budgets needed for Scenario 3 compared to Scenario 2.

- Scenario 4 & 5 have fixed budgets $100 mil/yr and $75 mil/yr and have 5% Poor constraint. Hence these two scenarios are different from first three scenarios as %Good varies year after year based on best possible combination of projects under the constraints.

- Scenario 2 (5% Poor and 20% Good) require total $747 mil for ten years. In comparison, Scenario 5 which is ($75 mil/yr) require $750 mil sum for ten years. These two scenarios hence have almost same total budgets for 10-year period and their mutual comparison is performed in bullet points below:
  - % Performance measures distribution (%Good/%Fair/%Poor) comparison between the two is discussed here. Scenario 2 performance targets distribution (Good 20%, Fair 75%, Poor 5%) and Scenario 5 has (Good 21%, Fair 77, Poor 3%). Scenario 5 provides marginally better performance as well as it is uniform annual budget-based scenario which is more realistic scenario.
Scenario 5 provides slightly better performance by the end of 10-year network average IRI (110) compared to Scenario 2 (115). Also looking at overall average IRI (average of annual averages) result is almost the same, Scenario 5 provides slightly better IRI (111) compared to Scenario 2 (114).

Scenario 3 (5% Poor, 28% Good) require total $1,047 mil for ten years. In comparison, Scenario 4 which is ($100 mil/yr) require $1,000 mil sum for ten years. These two scenarios hence have close total budgets for 10-year period and their mutual comparison is performed in bullet points below:

- % Performance measures distribution (%Good/%Fair/%Poor) comparison at the end of 10 year comparison between the two is also discussed here. Scenario 3 at the end of year 10 are estimated as (Good 28%, Fair 71%, Poor 1%) and Scenario 5 has (Good 22%, Fair 75%, Poor 3%). Scenario 3 seems to provide marginally better performance the annual budgets for this scenario are variable range from $77-$125 mil/year.

- % Performance measures distribution (%Good/%Fair/%Poor) comparison at the end of 10 years comparison between the two is also discussed here. Scenario 3 at the end of year 10 are estimated as (Good 28%, Fair 71%, Poor 1%) and Scenario 5 has (Good 22%, Fair 75%, Poor 3%). Scenario 3 has slightly better %Good (28%) compared to (26%) for Scenario 4, but Scenario 3 has higher %Poor (4%) compared to (1%) for Scenario 4. Hence, it is very hard to prefer one over the other for this comparison.

- Scenario 4 provides slightly better end of 10-year network average IRI (102) compared to Scenario 3 (104). Also looking at average IRI (average of annual averages) result is almost the same, Scenario 4 provides slightly better IRI (104) compared to Scenario 3 (108).

### 3.6 Moving forward

It has been demonstrated above that AgileAssets Pavement Analyst™ System can help roadway agencies to meet the FHWA MAP21-FAST requirements and establish pavement performance targets for their road network utilizing the following general methodology.

- Utilizing the multi-constraint Optimization Analysis:
  - Developing multiple network analysis scenarios
  - Set up appropriate objectives and constraint for each scenario
  - Running network analysis scenarios
  - Comparing Scenario results
  - Picking final targets and associated work program.
- Establish “Good, Poor, Fair” targets for overall pavement performance rating measure and/or individual condition metrics (IRI, Cracking, Rutting).
- Don’t get focused only on managing to the minimum condition %Poor threshold. Instead try different combination of Poor and Good distribution.
- Continue practicing good network pavement asset management and develop a variety of what-if scenarios which are innovative yet practical.

### 4.0 CONCLUSIONS

- Map-21 (FAST Act) performance measures can be used as constraints in the network analysis to determine costs to achieve desired performance targets.
- Comparison of alternative scenarios, with varying annual performance targets, is required to most effectively use available budget while maximizing condition goals and better distribute funds across years.
• Several what-if scenarios, that set performance/condition and budget constraints by treatment/budget categories may also need to be explored as well.

• Focusing on “Good” performance measure yields to spending on an optimal mix of different projects over the network aiming at lower total cost of the overall program (in long term)

REFERENCES


ABSTRACT:
Most state highway agencies rely on a pavement condition index to summarize overall pavement condition. These pavement condition indices in general only consider the pavement surface distresses and do not accurately account for the existing pavement structural condition. The Traffic Speed Deflectometer (TSD), being a continuous deflection measuring device, allows for the assessment of network-level pavement structural condition. Pavement sections with varying structural conditions can be identified and this information can be used to supplement the existing network-level pavement management decision-making process. This paper presents a framework to incorporate the pavement structural condition obtained from the TSD into a pavement management system (PMS) to enhance the network-level pavement management decision-making. An advantage of the proposed approach is that it links the structural condition to the existing pavement management decision-making process. This allows for relatively easy integration into the PMS.
USE OF PAVEMENT STRUCTURAL CONDITION DATA FROM TRAFFIC SPEED DEFLECTOMETER FOR NETWORK-LEVEL PAVEMENT MANAGEMENT DECISION-MAKING PROCESS

1. INTRODUCTION

Pavement condition indices are used to summarize the overall condition of pavement. Pavement condition indices are primarily based on a collection of different pavement distresses, such as rutting, cracking, roughness, etc. They usually range from 0 to 100, with 0 representing pavements that have completely failed and 100 representing almost new pavements free from any visible distresses. However, most of these distresses only account for the pavement surface condition and do not necessarily represent the actual overall structural condition of the pavement.

Pavement structural condition can be measured using both destructive and non-destructive studies, but currently deflection measurement is the only reliable non-destructive method for determining the structural strength of flexible pavements (Ferne et al. 2013). Traditionally, devices such as the Falling Weight Deflectometer (FWD) have been used to measure pavement structural condition. The FWD is a stationary deflection measuring device which imparts a load pulse to a pavement by dropping a weight on a circular load plate placed on the surface of the pavement. The FWD can only take stationary measurements at discrete points, limiting the density of points where data are collected. Moreover, because the FWD has to stop while taking measurements, it requires traffic control and can pose potential safety hazards to highway agency personnel and to the public. Consequently, the use of the FWD for network-level pavement structural condition evaluation has been limited.

State highway agencies rely on pavement management systems (PMSs) for selecting maintenance and rehabilitation treatments, determining the optimal time of these treatments, and estimating the future needs of the road network. Most PMSs do not use pavement structural condition information in their decision-making process. This is largely due to the limitations of stationary deflection measuring devices. But studies have shown that pavement structural information significantly affects the rate of pavement deterioration (Bryce et al. 2012, Flora 2009, Katicha et al. 2016). In addition, using pavement structural condition information in the PMS decision-making process could be cost-effective for state highway agencies (Zaghloul et al. 1998).

2. OBJECTIVE

In 2013, the Second Strategic Highway Research Program (SHRP 2) R06(F) project identified the Traffic Speed Deflectometer (TSD) as one potential device for network-level structural condition testing (Flintsch et al. 2013). The objective of this paper is to present an approach on using the pavement structural condition information from the TSD for network-level decision-making.

3. TRAFFIC SPEED DEFLECTOMETER

The TSD is a continuous deflection measuring device that can travel up to 60 mph and measure the pavement deflection velocity with a rear-axle load of 100 kN. The pavement deflection velocity is measured with the help of seven Doppler lasers positioned at 100, 200, 300, 600, 900, 1500, and 3500 mm from the rear axle of the TSD. The laser positioned at 3500 mm in front of the rear axle of the TSD, which is largely outside the deflection bowl, acts as a reference laser. The lasers are placed on a servo hydraulic beam to keep them at a constant height from the pavement surface. A climate control system maintains the trailer temperature at a constant 20°C (68°F) to prevent thermal distortion of the steel measurement beam. To remove the dependence of the pavement deflection velocity on the driving speed, the deflection velocity is divided by the instantaneous survey speed to give a measurement of deflection slope. The deflection measurement is then obtained by integrating the deflection slope measurements as follows:

\[ d(x) = \int_x^{\infty} s(y) \, dy, \]

where, \( s(y) \) = the deflection slope at location \( y \) measured from the applied load, and \( d(x) \) = the deflection at location \( x \) from the applied load.
In practice, the upper limit of integration should be selected large enough so that the deflection at that location is very small (practically zero); thus, the upper limit of deflection was set to 3.5 m (i.e., position of the reference laser). After obtaining the deflection measurements at different positions, various deflection indices can be calculated:

\[ SCI_{300} = D(0) - D(300) \]  
\[ DSI = D(100) - D(300) \]

where, \( D(0) \) = deflection at 0 mm from the rear axle; \( D(100) \) = deflection at 100 mm from the rear axle; \( D(300) \) = deflection at 300 mm from the rear axle; \( SCI_{300} \) = structural curvature index; and \( DSI \) = deflection slope index.

The effective structural number can also be calculated using Rhode’s equation (Rohde 1994):

\[ SN_{eff} = k_1 SIP k_2 H_p^k_3 \]

where, for asphalt pavements, \( k_1 = 0.4728 \), \( k_2 = -0.4810 \), \( k_3 = 0.7581 \) , and \( H_p \) = pavement thickness.

The structure index of the pavement (SIP) is calculated using the following equation:

\[ SIP = D(0) - D(1.5H_p) \]

where, \( D_{1.5H_p} \) = deflection at a lateral distance of 1.5 times the pavement depth.

4. FUNCTIONAL VS. STRUCTURAL CONDITION

The pavement functional condition is represented by pavement surface distresses, such as international roughness index (IRI), cracking, rutting, raveling, potholes, etc. Most states use a pavement condition index based on the pavement functional condition to represent the overall pavement condition. However, pavement functional condition does not necessarily represent the actual pavement structural condition.

The pavement surface condition data used in this study were obtained from two states: Pennsylvania and Virginia. Pennsylvania uses the Overall Pavement Index (OPI) to summarize the condition of pavement sections. The OPI ranges from 0 to 100, with 0 representing pavement that has completely failed and 100 representing a new pavement without any surface distresses.

Figure 1 shows the effective structural number calculated from the TSD and the OPI measured for the corresponding pavement sections in Pennsylvania. The pavement sections between 12 and 22 miles and 35 and 43 miles have a similar \( SN_{eff} \) of 3; however, they have significantly different surface conditions, with OPI values of 95 and 65, respectively. This shows that the observed pavement condition is not representative of the pavement structural condition measured by the TSD. A typical example of when the surface condition in the PMS and the structural condition obtained from the TSD can give different results would be a relatively structurally weak pavement section that has just been resurfaced with a 2-inch mill and overlay. Thus, the pavement structural condition obtained by the TSD can be used to identify pavement sections that are structurally different even though they have similar pavement surface conditions.
The Virginia Department of Transportation (VDOT) uses a surface condition index called the Critical Condition Index (CCI) to summarize the overall condition of their pavements. The CCI combines load-related and non-load-related distresses in an index between 0 and 100, with 0 representing pavement that has completely failed and 100 representing a new pavement without any surface distresses, similar to the OPI. Figure 2 shows the comparison between the SCI300 and 100 − CCI on a section of I-81 South in Virginia. The pavement section between mileposts 183 and 185 shows that the pavement section is significantly stronger compared to the surrounding sections (from mileposts 179 to 183 and 185 to 190), as represented by the low SCI300 values, but for the same section of the road the CCI values are comparatively lower. This indicates that the CCI does not necessarily represent the pavement structural condition. Another section between mileposts 205 and 210 has higher SCI300 values, representing a comparatively weaker pavement condition, but the same section has a very low CCI value, representing a pavement section with almost no visible distresses. Thus, the pavement functional condition does not always necessarily represent the pavement structural condition. This is more common now as most state highway agencies have moved towards aggressive preservation programs that decouple the pavement surface and structural conditions. The pavement preservation programs are mostly focused on the surface condition and do not necessarily contribute significantly towards structural health, though they can slow the structural decline of the pavements.
5. IDENTIFICATION OF SECTIONS WITH DIFFERENT STRUCTURAL CONDITIONS

Figure 3 shows a plot of the SCI300 computed from the TSD deflection measurements for the corresponding section of the road. The TSD was able to differentiate between pavement sections with different structural conditions. The bridges along these routes were significantly stiffer compared to the surrounding pavement sections; this could be observed from the low SCI300 values calculated from the TSD measurements. The TSD was able to identify most of the bridges that were structurally different compared to the surrounding pavement sections.

![Figure 3 Identification of bridges in I-580 North in Nevada (left) and I-85 South in South Carolina (right).](image)

Figure 3 Identification of bridges in I-580 North in Nevada (left) and I-85 South in South Carolina (right).

Figure 4 compares two plots of SCI300 measured on I-580 South in Nevada in 2014 and 2015, respectively. It can be seen that the TSD was able to identify a comparatively weaker section between 6 and 8 miles consistently for both years. Further studies have also shown that the TSD is capable of identifying pavement sections with different structural conditions compared to surrounding pavement sections (Katicha et al. 2013, Shrestha et al. 2018b, Uddin Ahmed Zihan et al. 2018).

![Figure 4 Pavement section with varying structural condition.](image)

Figure 4 Pavement section with varying structural condition.

6. FUNCTIONAL AND STRUCTURAL CONDITION INFORMATION TOGETHER FOR DECISION-MAKING

Most state highway agencies only consider the pavement functional condition in their PMS for the decision-making process. However, some states have started using additional information to improve their decision-making. For example, VDOT uses a two-step process that incorporates the traffic level, pavement structural condition, and the pavement age into the typical PMS decision-making, as shown in Figure 5 (Chowdhury 2008). VDOT uses an FWD to collect the pavement structural condition information. Since network-level data collection using an FWD is time-consuming, Virginia only carries out the test at nearly 5-year intervals.
Figure 5 VDOT two-phase decision-making process (Chowdhury 2008).

Utilizing deflection indices from the TSD could lead to more accurate information on the pavement structural condition as testing could be performed more frequently compared to the FWD. Deflection thresholds can be developed for deflection indices to categorize the pavement sections into good, fair, and poor structural condition (Rada et al. 2016, Shrestha et al. 2018a). Figure 6 presents an example of an approach that uses the pavement structural condition data to modify the decisions based on the functional condition of the pavement. Treatments are assigned to different pavement sections based on the CCI. These decisions are then filtered with the structural condition obtained from the TSD. For example, a pavement with a CCI score of 55 would be assigned corrective maintenance based on only the CCI value. When the structural condition is also considered, the decision-making branches as follows. If the structural condition is good and fair, the treatment would remain the same. If the structural condition is poor, the pavement would be recommended for rehabilitative maintenance.

Figure 6 Functional and structural condition together for decision-making.

Another approach is to develop a relationship between the remaining fatigue life of a pavement and the structural index computed from the TSD. Figure 7 shows an approach to link the deflection slope index (DSI) to the remaining pavement fatigue life. Relationships between the maximum fatigue strain and structural indices such as SCI300 and DSI have been developed for different pavement thickness (Rada et al. 2016). The strain values can be used to estimate pavement fatigue life (i.e., number of repetitions to failure; Huang 1993). Similarly, the relationship between the TSD-calculated strain values and pavement fatigue life can be developed as illustrated in Figure 8. Thus, the remaining
The service life of a pavement section can be estimated using deflection indices obtained from the TSD. Based on the remaining service life, the pavements can be categorized into good, fair, and poor, and used with the existing PMS decision-making as shown in Figure 6. Also, different pavement maintenance treatments can be recommended based on the fatigue life of the pavement. A decision matrix could be developed with the pavement surface and structural condition together, which would enable decision makers to make more informed and effective pavement management decisions.

Figure 7 Link between pavement fatigue life and structural index obtained from the TSD.

Figure 8 Fatigue life curves for TSD structural index DSI.

7. CONCLUSION
The main conclusions of the study are as follows:
i. The pavement surface and structural conditions do not necessarily represent each other. A pavement can have a perfect pavement condition index score and be in poor structural condition and vice versa. Thus, pavement structural condition should also be considered during decision-making based on the pavement condition.

ii. The TSD, being a continuous deflection measuring device, was able to identify sections that are significantly different based on their structural conditions.

iii. Two approaches were presented to implement the TSD deflection index into the pavement management decision-making process. The first approach involved using the pavement structural condition obtained from the TSD together with the functional condition to make effective pavement management decisions. The second approach presented a way to develop a relationship between the deflection index computed from the TSD and pavement fatigue life.

8. ACKNOWLEDGEMENTS
This study used the data collected during the Federal Highway Administration (FHWA) pooled fund project “Demonstration of Network Level Pavement Structural Evaluation with Traffic Speed Deflectometer” (DTFH61-11-D-00009-T-13008). We would like to acknowledge Greenwood Engineering for collecting and providing the TSD.
data, and FHWA for the guidance. Finally, we would also like to thank all the states that participated in this study. The paper does not necessarily represent the perspective of the agencies and organizations involved in the project.

REFERENCES


Chowdhury, T. (2008) *Supporting document for the development and enhancement of the pavement maintenance decision matrices used in the needs-based analysis.* Virginia Dept. of Transportation, Maintenance Division, Richmond, VA.


A transportation infrastructure with a high degree of coverage and quality is key to promoting sustainable economic development and improving the quality of life. In particular, road infrastructure is not only an important engine of development, but also one of the most valuable assets of the public sector. However, in Latin America and the Caribbean, road assets have often been neglected, and there is an important gap between the expectations and the needs of the general population and industry and the level of service offered by the road networks. One solution to reduce the infrastructure gap is the adoption of modern asset management practices that consider the total lifecycle of the roads, optimize road investments, and align those investments with sustainable development objectives. Effective and efficient management can help achieve the maximum benefits of these investments and avoid increased future costs caused by the lack of action at the most appropriate time.

This presentation will discuss the results of a study to determine the state of the implementation of road asset management in Latin America, assess the level of maturity of the asset management practices in the region, and compare these practices with the best practices globally. The main finding of the study is that there is a significant variation in the level of maturity among the roadway agencies in the region. Approximately one-fourth of the countries surveyed show characteristics that correspond to a competent or advanced asset management level, about half of the countries surveyed are at a basic level of asset management, and another one-fourth appear to be lagging behind and have not even defined an asset management strategy. In general, all the agencies recognize the importance of sound asset management and are moving toward advancing their practices. The presentation will also propose strategies on how to improve asset management practices and provide a basic roadmap to advance asset management practice in the region.
A Model Development for Prioritization and Optimization of Pavement Maintenance for Provincial Road Networks

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**KEYWORDS:**  
Pavement maintenance, Optimization, Prioritization Pavement management systems  

**ABSTRACT:**  
Most current pavement management system cannot be customized to reflect the local conditions. Thus, the identification of new approaches, which have been suited for the relevant factors in pavement management, is a major requirement.

The study is focused on determining priority weights of different factors used in pavement maintenance priority ranking and identify the main constraints that affect for the maintenance strategy in the provincial road agencies. Main constraints and priority factors were identified by the opinion survey. Based on the opinion survey five main priority factors were finalized namely Pavement condition, traffic volume, Connections to existing roads, Land use pattern and Importance to community.

Optimization model was developed to maximize the overall network condition with budget constraints. Priority index of each pavement sections is incorporated to optimization model. The proposed model is capable of determining the optimum maintenance activities for each road sections with priority indexes. Maintenance optimization is formulated as a linear integer programming problem. Applicability of the proposed model is illustrated using a case study and proposed model can be introduced as effective system to be used to select road sections for maintenance planning.
A MODEL DEVELOPMENT FOR PRIORITIZATION AND OPTIMIZATION OF PAVEMENT MAINTENANCE FOR PROVINCIAL ROAD NETWORKS

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1 INTRODUCTION

Over the years, pavement management systems have been utilized in road agencies to enhance maintenance management activities, provide the information needed to support the decision-making process and compare long-term impacts on alternative strategies. Pavement management is a fundamental part of the asset management of the road agencies and an important tool for large-scale investment in the economic and effective management. Each pavement maintenance system aims to increase the efficiency of repairing and treatment of pavements through utilizing the existing funds in their maximum benefits.

Considering the constraints faced by road authorities due limited capital allocation, technical and manpower limitation and socio-economic factors, maintaining the road network has become a very complex task. Lack of funding and other resources leads to poor maintenance level of the roads and lack of consideration of the socio-economic factors will result in improper prioritization of the maintenance activities. This is especially valid in the provincial and local road networks. Therefore, a systematic approach is needed to plan the maintenance program of the road network, taking into consideration the budget and other resource constraints and socio-economic factors in the area. Moreover, the pavement maintenance management system should be implementable at the provincial level, taking into account the resource constraints prevailing in those institutions. Methodology of selecting maintenance strategies for road authorities is an integral component of the pavement management system. Most of the current systems cannot be customized to reflect the local conditions with resources availability. It is required extensive data collection and calibration, which are not sustainable for those authorities especially in developing countries.

In this study Analytical Hierarchy Process (AHP) was used as the prioritization approach. AHP is one of the most important methods for prioritizing road maintenance factors since alternatives are very high in ranking process. Prioritization model was developed using five main criteria and sub criteria. Main criteria based on five specific factors which were relevant to local context in ranking pavement sections for maintenance. Pairwise comparison between each factor was done by engineers of road agencies and relative weights of each factor and rate of consistency of comparisons were calculated. Priority index for each factor was developed using relative weights and ratings for all road sections.

Optimization model was developed through application of linear programming methodology to maximize the pavement performance of the road network associated with budget constraints. Different maintenance operations for each road section were considered to optimize the maintenance process.

Basic Principle of Analytic Hierarchy Process

Analytic Hierarchy Process (AHP) is a mathematical technique used in multi-criteria decision making to help decision makers choose the best choice. In this approach, the complexity of the problem can be reduced by dealing with this complexity at different levels. Each level consists of a set of parameters with similar characteristics. In this approach, the overall goal is to be at the top level, followed by a set of criteria in the middle level, followed by a set of alternatives to achieve the overall goal. In general, criteria’s are further divided into sub-criteria based on the complexity of the problem. AHP recommends using a nine-point scale to calculate the relative importance of all elements, comparing them in pairs. Following steps are carried out to calculate priority value in AHP method:

a) Decompose the problem into a hierarchy of goal, criteria and sub criteria
b) Pair wise comparison of criteria and sub criteria using ratio scale
c) Organize the pair wise comparisons into a square matrix
   If the value of element (i,j) is greater than one, the factor in the i th row is important than the factor in the j th column and (j,i) element of the matrix is the reciprocal of the (i,j) element. Diagonal elements of the matrix are one.
d) Construct normalized pairwise matrix and Criteria weight vector
e) Evaluate the consistency of results by Consistency Ratio (CR)
If the value of CR is smaller than or equal 10%, the consistency is acceptable. If value is greater than 10%, it is needed to revise the subjective judgment. Ability of AHP to test the consistency of the results is one of the methods greatest important.

Methodology and Data Collection

In order to carry out the study it was necessary to identify the major factors relevant to the prioritization of road sections maintenance. The main objective of this study is to develop the maintenance strategy for rural roads network. It is proposed to select maintenance activities to be carried out on different rural road sections in two stages. Stage I determines rural road section priority based on the basis of the strategy proposed in this study. Stage II determines priority of activities on different sections. Thus, the strategy proposes that first sections which are more critical for maintenance needs to be selected. The strategy identifies to select maintenance activities using minimal data. Further, strategy proposes that the sections identified in stage I needs to be evaluated in more details so that the various maintenance activities to be carried out on these sections can be prioritized. Thus, the proposed strategy will be more economical as detailed studies needs not to be carried out on all the sections.

Identification of Priority Factors

Priority factors were identified by the opinion survey from the Engineers of provincial road agencies and through a comprehensive literature survey. Based on the opinion survey five main priority factors were finalized namely pavement condition, traffic volume, connectivity to local road network, land use pattern and importance to community. In order to quantify the Importance to community factor it was spitted into 4 sub factors namely, civic centers, cultural events, produces in area and alternative roads during maintenance.

<table>
<thead>
<tr>
<th>No.</th>
<th>Criteria</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Pavement Condition</td>
<td>Considering a present state of the road by using an index</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(e.g.: PCI -Pavement condition index , IRI - International roughness index)</td>
</tr>
<tr>
<td>2.</td>
<td>Traffic Volume</td>
<td>Consider bout the Number of vehicles per day</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(e.g.: Average daily traffic)</td>
</tr>
<tr>
<td>3.</td>
<td>Connections to existing roads</td>
<td>A or B Class</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C or D Class</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Pradeshiya sabha road</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Minor road</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dead end</td>
</tr>
<tr>
<td>4.</td>
<td>Land use pattern</td>
<td>Following factors are considered under this criterion</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Residential, Commercial, Industrial, Agricultural, Forest</td>
</tr>
<tr>
<td>5.</td>
<td>Importance to community</td>
<td>a) Civic centers served by the road (e.g.: schools, hospitals, post office, banks and temples)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>b) Cultural events served by the road (e.g.: Perahara and Priest)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>c) Produces in the area served by the road (e.g.: paddy, vegetable, tea and fruits)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>d) Alternative roads during maintenance</td>
</tr>
</tbody>
</table>
Rank the Importance of Selected Factors

Engineers of the road maintenance agencies were asked to rank the importance of selected priority factors. Each Engineer was approached individually 3 times within a month. Ranking was done by using AHP method which is used the ratio scale. Pair wise comparison matrices were checked for consistency using consistency ratio.

Hierarchy Structure of Goal, Criteria and Sub Criteria

![Hierarchy Structure of Pavement Maintenance Prioritization](image)

Figure 1. Hierarchy Structure of Pavement Maintenance Prioritization

Formulation of Prioritization Model

\[
P_I_s = \sum_{i=1}^{5} W_i F_{is}
\]  

(1)

\(P_I_s\) = Priority index of road section s (out of 100)
\(W_i\) = Priority Weight of factor \(F_i\)
\(F_{is}\) = Value of priority factor i for road section s (out of 100)

Priority weight of each factor was determined by the AHP method as described in previous section. \(F_{is}\) is the value assigned by the Engineers of provincial road agencies for each priority factor for all road sections. Scale use to assign this value is 0 to 100 and higher value represents the higher priority. \(P_I_s\) were defined by the summation of the product of \(W_i\) and \(F_{is}\).
Pavement Maintenance Optimization Model

Prioritization of maintenance activities depends on several factors such as present condition of road i.e. quantity and quality of deterioration, increasing rate of deterioration, importance of the different sections, etc. Hence, it is difficult to select various activities in order of their maintenance priority in a road network. Thus, there is an urgent need to develop a rational strategy for priority of maintenance activities to be carried out in a low volume road network.

Formulation of Optimization Model

Formulation of optimization model is carried out in such a way that the performance level of the road network is maximized with a cost effective maintenance strategy. Objective functions and constraints are described as follows.

Definition of Sets

- $R$: a Set of Operations (1-Do nothing, 2-Non-structural maintenance, 3-Minor rehabilitation, 4-Medium rehabilitation, 5-Major rehabilitation)
- $S$: a Set of Pavement Sections (1, 2, 3, 4 …12)

Definition of Parameters

- $L_s$: Length of road section $s$
- $T_r$: Cost of operation $r$ (per km)
- $C_{rs}$: Cost of applying operation $r$ to road section $s$
- $Q_{bs}$: Present condition of road section $s$
- $Q_{rs}$: Condition of road section $s$ after applying the operation $r$
- $P_s$: Priority index of road section $s$
- $B$: Total budget available for the year
- $Q_{min}$: Warning level for pavement condition
- $Q_{max}$: Maximum value of pavement condition

Definition of Decision Variables

- $X_{rs}$: 1 when operation $r$ is applied to road section $s$
- : 0 when no action applied

Pavement Performance Optimization Model

Maximize the Pavement Condition

$$\text{Maximize } Z = \sum_{r=1}^{R} Q_{rs}.P_s.L_s.X_{rs}$$  \hspace{1cm} (2)

Annual Budget Constraint

$$\sum_{s=1}^{S} \sum_{r=1}^{R} T_r.L_s.X_{rs} \leq B$$  \hspace{1cm} (3)
Annual Operations Constraint

\[ \sum_{r=1}^{R} X_{rs} = 1 \]  \hspace{1cm} (4)

Warning Level Constraint

\[ Q_{max} \geq Q_{rs} \geq Q_{min} \]  \hspace{1cm} (5)

Decision variable constraint

\[ X_{rs} \in \{0,1\} \]  \hspace{1cm} (6)

Objective function of maximization of performance of the road network as shown in Equation (2) was defined by summation of the pavement condition after the maintenance operations applied. The computed Priority index PI in equation (1) is incorporated in to the optimization model. Each maintenance action is accounted in the performance of the pavement by the objective function. Condition after applying each operation is defined as the following equation.

\[ Q_{rs} = Q_{bs} + \Delta Q_{rs} \]  \hspace{1cm} (7)

Condition after applying operation \( r \) is equal to the sum of present condition and condition increment after applying operation \( r \). First Constraint of the optimization model which is given in equation (3) ensures that maintenance expenditure does not exceed the available budget allocation. Only one operation could be applied for a particular road section during a particular optimization period as given in Equation (4). The maintenance actions should be carried out in such a way that road condition of each section after rehabilitation is above a minimum acceptable level as explained in equation (5). Equation (6) defines the decision variable \( X_{rs} \) to be an integer of value either 0 or 1.

Condition of the road section is measured by International Roughness Index (IRI), Pavement Condition Index (PCI) or any Index which can be measured by available data about road sections.

**Analysis and Results**

A hypothetical network of 12 different road sections was considered. Each pavement section was defined using section ID. The proposed strategy consists with two stages. Evaluation of priority index of each road section was done as the first stage of the analysis. Stage II includes the determination of maintenance operations for each road section by using developed optimization model.

**Analysis and Results for Stage I**

Priority index for all 12 road sections was calculated by using formulated prioritization model as explained in Equation 1. Priority weight for Pavement Condition, Traffic Volume, Connections to existing roads, Land use pattern and Importance to community were determined by the AHP method as explained below.

**Pair Wise Comparison of Criterion and Sub Criterion**

In this step pair wise comparison of Criterion of the decision hierarchy model was done by the road agency Engineers’ rankings. As presented in the hierarchy structure (Figure 1) Importance to Community and Connectivity to local road network have sub criterion.
Compute Normalized Matrices with Criterion and Sub Criterion Weight Vectors

In this step sum of the columns in reciprocal matrix is computed. Then each value of reciprocal matrix is divided by the column sum to find normalized vector and weights. Results shown in table 2, 3 and 4

Priority weights of each factor obtained by the AHP method are shown in table 2. It is clear that Importance to community factor has the highest weight when compare to the other factors. Land use pattern is the least important factor. Consistency ratio determines how consistent obtained results are. Consistency ration of this comparison is 0.089 which is lesser than 0.1 hence the results were accepted.

Table 2. Normalized Matrix with weights of Criterion

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Pavement Condition</th>
<th>Traffic Volume</th>
<th>Connectivity to local road network</th>
<th>Land Use Pattern</th>
<th>Importance to community</th>
<th>Total</th>
<th>Average</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pavement Condition</td>
<td>1 0.146</td>
<td>0.475</td>
<td>0.366</td>
<td>0.158</td>
<td>0.075</td>
<td>1.220</td>
<td>0.244</td>
<td>24.39</td>
</tr>
<tr>
<td>Traffic Volume</td>
<td>2 0.029</td>
<td>0.095</td>
<td>0.366</td>
<td>0.263</td>
<td>0.094</td>
<td>0.847</td>
<td>0.169</td>
<td>16.95</td>
</tr>
<tr>
<td>Connectivity to local road network</td>
<td>3 0.049</td>
<td>0.032</td>
<td>0.122</td>
<td>0.263</td>
<td>0.377</td>
<td>0.843</td>
<td>0.169</td>
<td>16.85</td>
</tr>
<tr>
<td>Land Use Pattern</td>
<td>4 0.049</td>
<td>0.019</td>
<td>0.024</td>
<td>0.053</td>
<td>0.075</td>
<td>0.220</td>
<td>0.044</td>
<td>4.40</td>
</tr>
<tr>
<td>Importance to community</td>
<td>5 0.728</td>
<td>0.380</td>
<td>0.122</td>
<td>0.263</td>
<td>0.377</td>
<td>1.870</td>
<td>0.374</td>
<td>37.41</td>
</tr>
</tbody>
</table>

Consistency ratio = 0.089

Table 3. Normalized Matrix with weights of sub criterion Importance to community

<table>
<thead>
<tr>
<th>Sub Criteria</th>
<th>Civic Centers</th>
<th>Cultural events</th>
<th>Produces in area</th>
<th>Alternative roads during maintenance Total</th>
<th>Average</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Civic Centers</td>
<td>1 0.571</td>
<td>0.749</td>
<td>0.286</td>
<td>0.381</td>
<td>1.987</td>
<td>0.497</td>
</tr>
<tr>
<td>Cultural events</td>
<td>2 0.143</td>
<td>0.187</td>
<td>0.500</td>
<td>0.476</td>
<td>1.306</td>
<td>0.327</td>
</tr>
<tr>
<td>Produces in area</td>
<td>3 0.143</td>
<td>0.027</td>
<td>0.071</td>
<td>0.048</td>
<td>0.289</td>
<td>0.072</td>
</tr>
<tr>
<td>Alternative road during maintenance</td>
<td>4 0.143</td>
<td>0.037</td>
<td>0.143</td>
<td>0.095</td>
<td>0.418</td>
<td>0.105</td>
</tr>
</tbody>
</table>

Consistency ratio = 0.043
Consistency ratio = 0.072

Importance to community and Connections to existing roads factors were divided into sub criterion and evaluated the priority weight of each sub criterion as shown in table 3 and table 4. Civic center has the highest priority weight under the importance to community criterion. Cultural events also have a significant priority weight when compare to the remaining factors.

Analysis and Results for Stage II

By using the priority weights of the criterions Priority index for each pavement section is computed and prioritized pavement sections feed to optimization model and maintenance operations are generated by optimization model.

Main input parameters of the optimization model include priority index values of road sections obtained from the previous analysis, length and PCI of each road section, Minimum expected PCI value, Annual budget and cost of each operation applied.
Table 6. Optimized maintenance operations for different budget levels with priority factor

<table>
<thead>
<tr>
<th>Budget %</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
<th>12</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>5</td>
<td>5</td>
<td>4</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>90</td>
<td>5</td>
<td>5</td>
<td>1</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>3</td>
<td>5</td>
<td>5</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>80</td>
<td>5</td>
<td>5</td>
<td>1</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>4</td>
<td>3</td>
<td>5</td>
<td>5</td>
<td>3</td>
</tr>
<tr>
<td>75</td>
<td>5</td>
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Table 7. Optimized maintenance operations for different budget levels without priority factor

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Note: 1- Do nothing, 2-Non-structural maintenance, 3- Minor rehabilitation, 4-Medium rehabilitation, 5-Major rehabilitation

The maintenance operations program obtained for the road network keeping minimum expected level of the each road section at a PCI value of 45 and assigning Priority index for all road sections is shown in table 6. Different maintenance strategies were obtained by changing the available annual budget. Maintenance operations were presented with respect to the different percentages of total budget. 100% of the budget is enough to maintain all road sections to achieve the maximum of overall network condition. Table values clearly show how the operations were changed when the available budget decreases. 50% of the road sections were not selected to apply any maintenance operation when the budget decreased to 10% of the total budget. In order to achieve the objective of maximization of total pavement condition while
considering the budget constraints condition of each road section vary with budget. Selected operations were oscillating without decreasing according to the budget.

Maintenance operations program obtained for the road network without assigning Priority index for all road sections (non-prioritized scheme) and keeping minimum expected level of each road section at a PCI value of 45 is shown in table 7. It shows the similar variation but different operations due to non-prioritized scheme.

This analysis is performed to illustrate the differences in improvements of the road sections caused by changing the magnitudes of the available budget. Maintenance operations depend on the priority index of each road section.

Table 8. Road sections condition after improvement with priority index

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It is clearly shows that for all budget levels, all road sections condition after applying operations is greater than or equal to 45 which satisfied the minimum expected level of road condition constraint. When the budget decreased, condition of the all road sections after applying operations were decreased drastically.

Table 9. Selection of road sections for operations

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0-Do nothing 1- Select for maintenance

Table 9 represents the comparison of operation selection for each road with respect to different budget levels. It is clearly shows that when the budget level is increased number of roads sections for maintenance also increased.
Variation of Total Pavement Condition

Figure 2 represents the comparison of total pavement condition for different budget levels corresponding to priority and non-priority scheme of road sections.

![Figure 2. Comparison of total pavement Condition with priority scheme and non-priority scheme.](image)

Total pavement condition decreases drastically when budget level decreases for both prioritized and non-prioritized schemes. Total pavement condition for both schemes is similar for 100%, 40% and 10% of the budget levels. Total pavement condition is always higher or equal for non-prioritization scheme when compare to the prioritization scheme. This analysis is performed to illustrate the variation of pavement condition improvement with respect to present pavement condition and priority index. Present condition of all selected road sections is below the minimum expected level of condition.

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Condition of each road section after applying operations is greater than the expected minimum level of condition. Compare to the improvement of road section 11 all other road sections improvement tends to maximum when the budget level decreasing. Priority index mainly affect to this variation.

**Effects of Priority Index for Maintenance Operations**

Figure 3 shows the new condition of road section 11 which has the minimum priority index among 12 road sections. Present condition of the road section 11 is 40. It is clear that improvement is considerably high under the non priority scheme (all sections have equal priority) for higher budget levels when compare to the priority scheme improvement. This implies that the incorporation of priority index to the optimization model gives the practical solutions.

![Figure 3](image3.png)

*Figure 3. New condition of section 11 - Qbs- 40, PIs-10*

Figure 4 shows the new condition of road section 10 which has higher priority index among 12 road sections. Present condition of the road section 10 is 35. It is clear that improvement between two schemes priority and the non priority scheme have similar improvement values for higher budget levels.

![Figure 4](image4.png)

*Figure 0. New condition of section 10 - Qbs- 35, PIs-90*
Figure 5 shows the new condition of road section 7 which has the higher priority index and higher present condition among 12 road sections. It is clear that improvement between two schemes priority and the non-priority scheme have similar improvement values for all budget levels.

Figure 5. New condition of section 7 - Qbs-85, PIs-90

Figure 6 shows the new condition of road section 3 which has the minimum priority index and higher present condition among 12 road sections. Present condition of the road section 11 is 95. It is clear that improvements are comparatively similar when the present condition is high and the priority index is low.

Figure 6. New condition of section 3 - Qbs-95, PIs-10
Conclusion and Recommendation

Pavement management is a very complex task, if it’s considered as a whole. In order to find the best strategy for providing, assessing and maintaining an acceptable level of service during a pre-selected time period, an effective road management plan should be identified and the resources should be allocated properly.

In this study, a combined approach of prioritization and optimization for road maintenance planning has been introduced. The proposed method generates an optimal maintenance plan for a road network while considering the priority weight of each road section.

Inconsistency in judgments is one of major limitations of prioritization methods used in pavement maintenance planning to prioritize pavement maintenance activities. In order to overcome the limitations associated with ordinary subjective prioritization, it is necessary to determine a reasonable procedure for assessing maintenance priorities. Through the comprehensive literature review of prioritization approaches, based on the operational advantages of the AHP approach when dealing with a large number of projects and its ability to generate priority assessments, the AHP approach is considered the preferred approach for prioritizing road maintenance.

For the purpose of prioritization of road sections five main prioritization factors were found by road agencies’ Engineers opinion survey. It was found that a higher weight was given to importance to community factor followed by pavement condition, traffic volume, connectivity to road network and land use pattern. Under the importance to community, Civic centers have the highest priority. Cultural events also have the significance priority since the study was done for provincial road network. Consistency ratio for all criterions is less than 0.1 and it’s accepted the results of AHP.

The Priority index that is computed for different roads is incorporated in to the optimization model. The importance of incorporating priority index which is based on several non-technical factors is that it allows the road agencies to consider the needs of the communities without totally neglecting them. This will increase the acceptance of such a system for maintenance planning in provincial road agencies, where often maintenance decision making is often highly ad-hoc and politicized.

Maintenance strategies obtained by reducing the available annual budget were oscillating without decreasing according to the budget change. Improvement is considerably high for lower priority sections under the non priority scheme when compare to the priority scheme improvement. This implies that the incorporation of priority index to the optimization model gives the practical solutions. Maintenance operations program obtained for the road network without assigning Priority index for all road sections (non prioritized scheme) shows the similar variation but different operations due to non prioritized scheme.

The developed model does not require extensive data collection as the type of pavement condition of road sections can be defined by the user; hence it can vary from a PCI derived from detailed distress surveys, IRI or simple qualitative rating of the pavement condition. Therefore the proposed model can introduced as effective.

Other factors such as road safety, road upgrading priority can also be incorporated into the prioritization model. Road user cost function can be added for the optimization model and road maintenance cost can be included as a function of road condition. They can be considered as recommendations to the developed model.
References


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Farhan J (2011),Integrated Prioritization and optimization Approach for Pavement Management


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Lessons Learned in Rehabilitating Liberia’s Road Network

Fifteen years ago, Liberia emerged from its long civil wars with its road network in ruins. Today, it has a paved roads network extending outwards from the capital and a growing network of improved secondary and tertiary roads. This transformation came despite having funding come from a variety of projects funded by different donors each of which has their own priorities and tended to focus on their own timelines.

Background

Liberia is on the southern coast of West Africa. Its topography ranges from coastal lowlands with mangroves and swamps, to rolling hills and plateaus further inland, and low mountains in the northeast. Annual precipitation varies from over 4000 mm in the coastal region down to 1500 mm in the higher elevation areas near the border with Guinea—Monrovia has the highest rainfall of any capital city.

Liberia’s rural road network was poorly developed even before the start of its civil war in 1980. Most rural roads were little more than unimproved tracks. Cross-drainage structures were generally locally built out of rough-hewn timbers. During the 25 years of fighting, very little work was done on these roads. When Liberia finally emerged from its wars, the all of its roads were in bad condition. During the heavy rainy season, many roads were completely impassable.

The Liberian Government stated that rebuilding its road network was one of its top priorities. The international community responded with a series of different projects, the most important being the following:

1. The World Bank funded the rehabilitation of urban streets in Monrovia, the rehabilitation of the roads from Monrovia to Buchanan (148 km) and Monrovia to the Guinea Border (246 km).
2. African Development Bank funded the paving of the Fish Town-Harper Road (50 km).
3. USAID’s Liberia Community Investment Project (LCIP, 2007-2010) which included the repairs to 381 km of roads with the main focus being clearing the roads and building bridges.
4. Sida’s Liberia-Swedish Feeder Road Project, (2012-2016) rehabilitated 400 km of roads in Bong, Nimba, and Lofa Counties with a focus on the earthworks, building only a minimum number of culverts.
5. USAID’s Farm to Market Roads Program (2015-2020) included the rehabilitation of 300 km of rural road, including bridges with total spans of up to 10 m and the rehabilitation of the highway from Gbanga and Voinjama.

The focus of these projects has been on the central, more populated region of Liberia. The roads to the southern counties remain in very bad condition.

Lessons Learned

Each of the donor-funded projects had its unique challenges and issues, but four main lessons were common across the different interventions:

1. Start by creating access: Liberia had a weak road network before the civil war. Many of the bridges on the secondary and tertiary roads were locally fabricated wooden bridges and had collapsed during the wars. The first priority became rebuilding these structures. If a road was passable it would be used. Since the major form of transport is motorcycles, creating motorcycle tracks and bridges can allow for a rapid expansion of a passable network. Access became a critical concern during the Ebola Viral Disease outbreak when tracking down isolated cases became a priority.
2. Select roads for rehabilitation based on traffic volumes: The primary network should be rehabilitated from the main cities outward. Rural roads should be opened to basic access first and upgraded based on increased traffic. It can be very expensive to pave isolated sections of road as was seen with the rehabilitation of the Fish Town-Harper Road. Selecting rural roads to be upgraded based on estimated future demand is difficult. In both the Sida and USAID-funded rural road programs, some rehabilitated road sections are used almost exclusively by motorcycles.
3. Call it Rehabilitation, but upgrade the roads: Rehabilitation is defined as “the action of restoring something that has been damaged to its former condition.” Donors like this terms since it seems to imply that the
The environmental consequences of the work will be minor. However, the real goal is to upgrade what had been an engineered track and make it into a good road or two make a primary road into a safe, modern highway.

4. **Focus on building capacity, not just roads**: One of the biggest challenges that we have is finding good engineers. The same is true for the government, local contractors, and consultants. A three or four year capacity building program is not enough. It takes a decade of continued effort.

5. **Begin developing the funding for maintenance works early**: It has taken Liberia many years to develop a funding plan for the road maintenance and it is still not fully implemented. It involved developing and passing legislation, creating new institutions, and then getting the whole system to work. This process takes at least five years, so best to begin working on this issue early.
3D Pavement Analysis
Using a device made of lasers, cameras, and inertial measurement units, transportation organizations can create a 3D model of their road pavement network. This 3D model is capable of creating crack mapping, longitudinal profiling and digital terrain modeling. Many people perform these tasks by visual inspection, with boots on the ground and semi automation. The 3D process of collection has the ability to provide results through automation in a desktop environment with highly repeatable results. When analyzing an organizations network, this capability will reduce the time to get results, reduce the amount of human error in the process and give results that are accurate and repeatable.

In this presentation, we will discuss reasons behind the rapid growth of 3D Pavement Analysis and explore best practices for implementation

- What is 3D Pavement Analysis
- How is 3D Pavement Analysis performed
  - Equipment
  - Software

Advantages and Benefits of 3D Pavement Analysis
ROADROID SMARTPHONE TECHNOLOGY

ABSTRACT

Roadroid addresses the themes in “Asset Management and Innovative Technology” themes as well as Low Volume-Low Cost roads (Rural Roads).

Roadroid smartphone technology is the way of the future providing roughness surveys and data collections for sealed roads, unsealed roads and footpaths. The cloud based Swedish technology is very easy to use allowing large amounts of data to be collected and then analysed very quickly, with data being available within an hour of uploading.

This presentation will outline how the application works, data collection and many other uses of the Roadroid application. I will also showcase some exciting new features for rapid analysis of large amounts of data for making decisions based off best asset management practice. As data is easy to capture it can be used in operative maintenance - not only for annual planning.
ASSET MANAGEMENT & PRESERVATION

Asset Management & Preservation is the pillar of roads sustainability and durability. Proper and periodical maintenance optimize the actions and lowers the long-term costs. In the process, it’s fundamental to collect objective data about the road condition.

Building a road whether it is paved highway or a low volume unpaved road is always costly. Periodical maintenance is the key element for the roads sustainability. In long term, small maintenance costs can result in savings on the higher costs for re-construction and re-building the whole road.

Carrying out a road survey is not always easy and cost-effective. It requires professional skills, appropriate technology and devices. Consequently, the number of periodic survey of roads, and especially rural and low-volume roads, tends to be low.

Roadroid has enabled Road Organizations around the world to monitor and survey the road roughness (IRI) and overall condition of road networks in a very simple way. The systems low cost and simplicity makes it widely-available for Road Authorities to carryout otherwise time consuming and expensive road surveys with smartphones.

The solution to use the smartphones for road roughness condition survey and road inventory is a very efficient and cost-effective method in today’s world. It has helped the road agencies all over the world to outreach more roads and especially those rural and low volume roads. The system is easily accessible by our easy to use technologies and devices and doesn’t require professional expertise to operate and carry-out a survey.

During 2016 Roadroid have developed its mobile app for roughness, to capture also GPS-video. After the video is uploaded to its web service, it is possible to make road inventory from it, from the office. This enables the road organizations to carryout various road inventory surveys by visual details obtained from the video playback. This feature further preserves the cost, time and hassle to travel to site. This has a specific interest for low volume roads.

Using smartphones to survey a road also decreases the capital at risk, depreciation cost, security threats in insecure areas and transportation costs where sometimes it really is a burden to transport survey vehicles and devices.

Sorting out the other usages of mobile-based road survey solutions, we can see goods examples from developed countries where it is used on daily basis to monitor the road condition variation over a specific period of time or seasons, traffic count and external factors affecting the quality of the roads. In some cases, using mobile-based roads survey systems has resulted to create close to a real time road condition status platform where prompt information can be obtained any time.
Smartphone based survey solutions is also used in security risk-prone and or impassable remote areas to carry out a road condition survey where it involves a high risk to travel with expensive survey vehicles. A smartphone can be mounted safely inside a vehicle to carry out a survey supported by GPS coordination, photos and videos. The captured video can also be used back in the office later to do an inventory survey of the same road.
ROADROID MOBILE-BASED ROAD SURVEY INSTRUMENTS - OVERVIEW

1. Roadroid Pro App:

The Roadroid Pro app measures the road roughness (IRI – International Roughness Index) by using the smartphones accelerometer - and can automatically capture GPS-enabled photos or videos of the road.

There are complex formulas behind the signal analysis and the result is expressed in a global standard - International Roughness Index (IRI). So, every second an IRI value is saved with an X, Y and Z coordinate from the GPS. Data is stored on the phone and then transferred by Wi-Fi or 3G/4G to a cloud server.

After the data has been transferred to the cloud service it can be monitored on a map. The data is assigned 4 colors depending on the road condition. Green for good, Yellow for Satisfactory, Red for Unsatisfactory and Black for Poor. Data is also aggregated in 100-meter sections, and can easily be downloaded to create analyses and charts in Excel.

Roadroid is a response type survey system. It is according to the world banks Information Quality Level 3 (IQL3). A laser survey vehicle is IQL1. IQL3 gives about 80% accuracy in comparison to IQL1.

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 profilometric methods;
Class 3—IRI estimates from correlation equations;
Class 4—subjective ratings and uncalibrated measures.

Data collection with smartphone based systems like Roadroids, will not directly compete with Class 1 [1] precision profiles measurements, but instead, complement them in a powerful way. As Class 1 data are very expensive to collect, it cannot be done often. Beside this, advanced data collection systems also demand complex data analysis and take long time to deliver the result. With smart phone based data collection, it is possible to meet both these challenges. A smart phone based system is also an alternative to Class 4—subjective rating [1], on roads where heavy, complex and expensive equipment is impossible to use.

Research done by the University of Auckland in 2013¹ and the University of Pretoria in 2013² showed that Roadroid system is consistent enough during IRI measurements with varying

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vehicle conditions such as speed, road path, loads, tyre pressure etc. The Auckland study showed that Roadroid has an 81% correlation with laser measurement systems.

2. Roadroid Road Inventory:

The Road Inventory App for android smartphones registers manual-ocular inputs in 5 classification levels of a certain inventory parameter. Data is saved every second and the app uses the Androids GPS for Longitude, Latitude and Altitude of the data. You have available over 100 parameters ranging from average road condition to detailed cracking inventories.

FIGURE 3  Road inventory app being used to enter manual ocular inputs during a road inventory survey
Some identified benefits of The Road Inventory App compared to pen and paper

- Saves lots of time and increase security of field work. There is less need to have to stop along the road and take notes
- It is more detailed. Normally one would note the average state in 100-meter sections, with the Road Inventory app it is possible to register differences every second (every 10 meters in 36 km/h).
- Powerful and immediate visualization on maps. View collected layers in the web tool.
- Export as .txt files in 20-200-meter segments for direct import to HDM4 or other RMMS.

A recently added functions is that the inventory can be done from a prerecorded video by Roadroid Pro App, which will also save both time and money.

A simple illustration showing the method for doing road inventory survey from a pre-recorded video by Roadroid Pro App.

The Road inventory app is easy to use and highly portable - bring anywhere and run on battery for many hours. It’s also durable - and there are no expensive or rare spare parts.

3. Roadroid Event Manager

The Road Event Manager is an app that registers location and condition of culverts or damages i.e. washouts or complete bridge break-down, etc. where driving and passing may not be possible:

- The app receives coordinates from GPS.
- Add road section ID and choose from +20 categories to register.
- Add data to three data fields: Type, Registration and Action.
- Use the phones camera to capture GPS photos to the object.
- Possibility to add lane and direction data.
- Possibility to add condition/severity (1-5) of an eventual damage.
- State - can be modified through the web tool as it changes during the work progress.
- View the objects on a map – with photos.
- Summarize all your object data in a .txt file, for further work in MS Excel or RMMS.

New Zealand Application of Roadroid

Since 2015, Roadroid has carried out continuous daily surveys of low-volume gravel roads in Southland, New Zealand, to monitor the variance of road condition during the whole year period and also to assess the seasonal effects on the road network. We utilise Fonterra Diary tankers to carry out surveys for us as they cover 85% of our network on a daily basis (unsealed network of 3000km).

My presentation will cover other uses of Roadroid that we have been utilising the app for here in New Zealand such as: Deterioration monitoring, serious crash reviews, detour routes and driver behaviour monitoring.

I will also cover off in my presentation some smarts that have been developed to help analyse very large amounts of data very quickly. Upon implementing Roadroid, we found it was very easy to capture large amounts of data, but it was what we done with that data was the challenge – we now have the answer in the form of Change Reports. These reports effectively compare one timeframe with another and report back on change in roughness and change in operating speeds – from this hotspots can be identified and work programmes can be developed.

CONCLUSIONS

Measuring roads with smart phones can provide an efficient, scalable, and cost-effective way for road organizations to deliver road condition data. Results shows that Roadroid has universal usage; but with a specific potential on Low-volume/Low Cost Roads (Rural Roads).

Smart phone based gathering of roughness data and Road Inventory can be done at a low cost and 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 be advantageously used in performance based contracts or research on road deterioration, various environmental effects (as heavy rains, flooding, etc.) and other adjacent purposes.

The solution to use the smartphones for road roughness condition survey and road inventory is a very efficient and cost-effective method in today’s world. It has helped the road agencies all over the world to outreach more roads and especially those rural and low volume roads. The system is easily accessible by our easy to use technologies and devices and doesn’t require professional expertise to operate and carry-out a survey. With continuous data collection of the road it is also possible to get close to real time information about a road’s condition.
Using smartphones to survey a road also decreases the capital at risk, depreciation cost and transportation costs where sometimes it really is a burden to transport survey vehicles and devices.
The use of Geographic Information System (GIS) Technology & Satellite Imagery for the Assessment, Prioritisation, Engineering Design, & Supervision of Road Networks In Nigeria.

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1.0 INTRODUCTION
1.1 General
Almost everything happens somewhere and in most cases, knowing where some things happen is critically important.
Examples:
- Position of country boundaries
- Degree of road/bridge dilapidation
- Factors affecting settlements
- Management and supervision of Infrastructure
- Allocation of funds for projects

1.2 Objectives
After the paper presentation, attendees would have been exposed to the following:
   a. Introduction to GIS Technology
b. Assessment, Monitoring And Supervision Of Road Network/Projects Via GIS

c. Prioritization Of Road Network Via GIS As Against Political Prioritization

d. Assistance Of GIS In Engineering Designs Of Road Network

e. Benefits Conclusions and Recommendations Of GIS

1.3 What is (Geographical Information Systems) GIS?

Geographical Information Systems (GIS) is a computer-based system for showing, analyzing and modelling spatial data in a way that is much faster, easier and flexible than plotting information on traditional paper-based maps. (Longley et al, 2005)

GIS are a special class of information systems that keep track not only of events, activities, and things, but also of where these events, activities, and things happen or exist. Figure 1.1 below shows a pictorial description of GIS. (Longley et al, 2005)

Geographic location is an important attribute of activities, policies, strategies, and plans. Geographic problems involve an aspect of location, either in the information used to solve them, or in the solutions themselves. (Longley et al, 2005)

Fig. 1.1: Showing Geographic Information System (Overview of How it Works) (Source: souzand, 2016)

2.0 SPATIAL DATA, A BACKGROUND TO GIS

2.1 What is Spatial Data?

Spatial data, which is also known as geospatial data or geographic information is the data or information that identifies the geographic location of features and boundaries on Earth. It is usually stored as coordinates and can be mapped. Spatial data requires spatial literacy before it can be acquired. (Vangie, 2018)

2.2 What is Spatial Literacy?

- Spatial literacy is an important part of becoming a good geographer
- It involves a lot of different ideas and skills which come together to help you ‘read’, understand and use spatial materials such as maps
• Spatial literacy is also the ability to create spatial data by:
  (i) Drawing maps, diagrams and graphs
  (ii) Using more technologically advanced ways of drawing maps, diagrams and graphs like the use of Remote Sensing and Geographic Information Systems (GIS) (Edmed, 2016)

2.3 Examples of the Many Ways of Showing Data/Information Visually

2.3.1 Topological maps: These are maps that has country size related to the size of the data for that country (Edmed, 2016). It is a type of diagram that has been simplified so that only vital information remains and unnecessary detail has been removed. These maps lack scale, and distance and direction are subject to change and variation, but the relationship between points is maintained.

Fig. 2.1: Showing the Topological Map of the World (Source: Edmed, 2016)

2.3.2 Land-use maps: These are maps that has different shading for different types of land use, such as infrastructures, housing and shops (Edmed, 2016). City planners need to know which areas of a city are used for which purpose. Therefore, they produce a map of "land use", that identifies parts of a city and the major activities (land use) that happen there. Remote sensing imagery e.g. GIS is very useful for this purpose, since you certainly don't want to spend many weeks or months walking or driving around a city to map its land use. But to use remote sensing imagery effectively, you have to be able to interpret it accurately. A typical one is land use to counteract flooding. It can also be used to signify built up areas.

Fig. 2.2: Showing the Topological Map of an Area (Source: Edmed, 2016)
2.3.4 Dot maps: These are maps onto which dots are placed to show the location of data. A large portion of the new maps appearing today are generated not through government agencies or geographical societies and associations, but by common individuals from all walks of life. Users of such virtual globe services as Google Earth constantly create new thematic maps quickly and easily. Such services offer a base platform upon which users can display layers as they choose, or even add their own data to the display. In this case a dot for each 100,000 people found at a location (Edmed, 2016). Location of mangrove swamps and population distribution. This can be used to plan for Infrastructure and basic amenities for an area or country.
2.3.5 Choropleth maps: They are maps that use differences in shading, colouring, or the placing of symbols within predefined areas to indicate the average values of a particular quantity in those areas. Areas are shaded within data groupings to show spatial patterns. Choropleth maps provide an easy way to visualize how a measurement varies across a geographic area or show the level of variability within a region. It is a map of population change that can be used in analysing and planning for areas of concentration for Infrastructure.

Fig. 2.5: Showing a Dot Map that Signifies the Population in an Area. (Source: Edmed, 2016)

Fig. 2.6: Showing a Typical Choropleth Map Indicating the Most Populated and Less Populated Countries (Source: Edmed, 2016)
3.0 ASSESSMENT, MONITORING & SUPERVISION OF ROAD NETWORK

3.1 Assessment

3.1.1 How GIS works: Take for example, two sets of data put into a GIS package
   a. Data shows location of road traffic accidents
   b. Data shows local road network
   c. Two ‘layers’ can be brought together as a simple map to show where road network accidents are:
      • Could take this problem further by adding more data, e.g. time of day of accidents, weather conditions, etc.
      • Can be used to predict accident hotspots, likely impact of particular safety features such as speed ramps and roundabouts

Fig. 3.1: Showing Overlay of Information that Typifies How GIS Works (a) (Source: Edmed, 2016)

Fig. 3.2: Showing Overlay of Information that Typifies How GIS Works (b) (Source: souzand, 2016).
The Following image illustrates the capability of medium resolution satellite images in conducting visual analysis of data. In the project (Enugu State, Nigeria) it was intended to use the full resolution images that can provide better details needed for the project.

Fig. 3.3: Showing Overlay of Information of Enugu State, Nigeria (Topological Map, Boundary and Name of State)

3.2 Monitoring

3.2.1 How GIS works: In Delta State, Nigeria, the information about construction cost and the impact of each project on population and the likes were added to the GIS database. Decision makers can use the GIS to click on any road and immediately gather all information about each project. The following image illustrates this capability of the GIS that we have developed:

Fig. 3.4: Showing a GIS Data Base of Delta State that can be used to Make Infrastructural and Other Decisions
Figure 3.5 displays an overview of the entire ‘on-system’ highway network of the Fort Worth area, Texas USA and the user interface of the GIS-based tool that was developed. Given the scope of operations of this district, the need for seamless integration of individual information systems is evident. Hence, Fort Worth area is being properly monitored. (France-Mensah et al., 2017)

![Figure 3.5: Screenshot of User Interface Of GIS-Based Tool With On System Highway Network of Fort Worth, Texas (Source: France-Mensah et al., 2017)](image)

Figure 3.6 show visual example of conflicting project layers for projects across databases and road sections receiving repetitive annual Maintenance and Rehabilitation treatments respectively. This will in turn assess the focus of the Government on the roads and gather information on conflicts of road projects as presented in budgets, so as to safe repetitive expenses. (France-Mensah et al., 2017)

![Figure 3.6: Showing Conflicting Highway Projects on the same Pavement Section in fiscal year, 2017 (Source: France-Mensah et al., 2017.](image)
3.3 Supervision

3.3.1 How GIS Works: The role of GIS does not end at the design stage. It also continues into the supervision stages. We intend to use the GIS to enter for each structure all information related to quantities, BOQ, contractor progress, payment certificates, all other information related to supervision and construction activities. We have worked on other projects in other parts of the world where GIS technology was used as the database for monitoring the progress of construction work and for managing spatially distributed projects. Figure 3.7 shows how all river crossings in the riverine area.

Fig. 3.7: Showing How All River Crossings in the Riverine Area of Delta Were All Entered In GIS

Fig. 3.8: Showing Quantity of Contracts in Each Municipality and Details of the Progress of Works within Direct Influence Area in Brazil. (Source: souzand, 2016)
4.0 PRIORITIZATION OF ROAD NETWORK VIA GIS AS AGAINST POLITICAL PRIORITIZATION

4.1 Prioritization:

4.1.1 How GIS works: In the Delta State, Nigeria Project, a study of all necessary bridges was conducted. The purpose was to evaluate existing bridges and set a certain priority for which bridges are to be constructed first and which ones can come later. We have looked at each bridge from an overall transportation need within the state and by trying to meet other important political factors such as equity between different ethnic groups and geographic regions.

Fig. 4.1: Showing Evaluation of Existing Bridges and Priority Being Set for Which Bridges Are To Be Constructed First and Which Ones Can Come Later.

- Figure 4.2 also illustrates how all bridges and river crossings in Delta were entered into the GIS, prioritized, and assigned various information including construction cost and benefitting population.

Fig. 4.2: Showing Prioritization of Bridges Including Construction Cost and Benefitting Population.
5.0 ASSISTANCE OF GIS IN ENGINEERING DESIGNS OF ROAD NETWORK

5.1 Kwara, Nigeria as Case Study

As previously discussed, Geographic Information System gives excellent stages in achieving successful and functioning projects right from Assessment to Prioritization, thereby making it proactively possible to conceive workable and non-conflicting designs for road and bridge projects. In figure 5.1 and 5.2, a careful monitoring of Paiko, in Kwara state shows the dilapidating nature of a federal road that has degraded overtime from 2003 to 2017. Therefore, with the use of GIS, a search light had been turned on to help divert focus to this road and many others like this. This will in turn help the workability of Nigeria’s budget in relation to Road and Bridges Infrastructure. Furthermore, a proper reconnaissance survey can then be carried out by a team of Engineers so as to prepare a workable geometric design as well as the Bill of Engineering Measurement and Evaluation (BEME). This is how projects should be run.

Fig. 5.1: Showing Paiko, Kwara State with the Federal Road as at the Year 2003 (Source: google, 2018)

Fig. 5.2: Showing the Dilapidating Nature (Asphaltic to Lateritic) of the Road as at Year 2017 (Source: google, 2018)
5.2 Fort Worth, USA as Case Study

In Fort Worth, projects data were visualized as feature layers according to the county, project type, and fiscal year. Figure 5.3 shows a visual of the 4-years Project Management Planning (PMP) for the district according to the fiscal year as displayed in the tool. It also includes some projects from the 10-years UTP from the long-term plan for highway construction projects in Design and Construction Information System (DCIS). In addition to this, intra-database and inter-database analyses were also performed and added as layers to the tool. (France-Mensah et al, 2017)

Fig. 5.3: Showing DCIS projects by fiscal year and county for 4 years –plan (Source: France-Mensah et al., 2017)

5.3 Digital Elevation Model (DEM)

5.3.1 Introduction to DEM

Digital Elevation Model (DEM) is the representation of continuous elevation values over a topographic surface by a regular array of z-values, referenced to a common vertical datum. DEMs are typically used to represent the bare-earth terrain, void of vegetation and manmade features.

GIS can help define a road alignment from one point to another on a virgin road, this implies Digital Elevation Model.

5.3.2 Types of DEM

A DEM can be represented as a raster (a grid of squares, also known as a height map when representing elevation) or as a vector-based triangular irregular network (TIN). The TIN DEM dataset is also referred to as a primary (measured) DEM, whereas the Raster DEM is referred to as a secondary (computed) DEM. (Ronald,1987) The DEM could be acquired through techniques such as photogrammetry, lidar, IfSAR, land surveying, etc. (Li et al. 2005). DEMs are commonly built using data collected using remote sensing techniques, but they may also be built from land surveying. DEMs are used often in geographic information systems, and are the most common basis for digitally produced relief maps. While a DSM may be useful for landscape modeling, city modeling and visualization applications, a DTM is often required for flood or drainage modeling, land-use studies, (Balenoic, et al., 2015) geological applications, and other applications. (Glossary, 2007) See figures 5.3 and 5.4

Amidst all, GIS can also help to maneuver rocks and rivers when it’s been given clauses for establishment of alignments.
Fig. 5.4: Showing Digital Elevation Data (a) (Source: google, 2018)

- Brown – High Points/ Mountains
- Yellow – Rough/Irregular terrain
- Green – Flat terrain, Vegetation like Grasses, Shrubs and Trees
- Blue – Streams, River, Seas

Fig. 5.5: Showing Digital Elevation Data (b) (Source: google, 2018)
6.0 BENEFITS ANALYSIS, CONCLUSIONS AND RECOMMENDATIONS OF GIS

6.1 Benefits Analysis

GIS has proven its usefulness in many branches of Science and Engineering (Asgei, 2012). Numerous of examples of integration of GIS into real world activities have been published in peer-reviewed journals, ranging from simple map representations to power system analysis tools and energy conservation programs, which have shown direct benefits in terms of time, cost and performance. An example of implicit benefit of GIS is Google Earth, a system which provides base information of locations and related facts, and which users can integrate their own data for specific tasks.

6.2 Conclusions and Recommendations

GIS application made it possible to remotely monitor public works, allowing for analyses of multiple data points through graphs and thematic maps. Results of the experimental application were satisfactory, enabling the analysis of a large number of road and bridge constructions. Without IT/GIS resources, it would not have been possible to achieve the obtained results.

Staggering amounts of information are involved in the project delivery process. GIS offers the advantage that it can process and manage quantities of data far beyond the capabilities of manual systems or fragmented application programs dispersed throughout the organization and with stakeholders. Data are stored in a uniform, structured manner that facilitates data exchange and integration of ICT tools.

However, underutilization and interoperability issues have been identified as problems in GIS adoption. Generally, GIS is customized to serve the operational or business objectives of firms and governmental bodies and its applications are often not interoperable or compatible. To overcome these barriers, focus must be stretched over the (Spatial Data Infrastructure) SDI, where the concern should be to implement a framework of geographic data, metadata, users and tools that are interactively connected in order to use spatial data in an efficient and flexible way, as well as the redesign of processes.
REFERENCES


Edmed, David (2016, March 1), Introduction to GIS. Online Power Point Presentation.


**APPENDIX**

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PAPER TITLE  AI and Roadway Assessment

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ABSTRACT:  
Every public asset requires continuous maintenance. Optimal maintenance is easiest achieved with continuous, inexpensive, safe, and precise monitoring. For assets such as cars or jet engines, monitoring technology is well developed. Due in part to the size and complexity of road networks, monitoring technology for such networks is in its infancy. The current alternatives available to do this are any or all of subjective, non-scalable, unsafe, or very expensive. What is needed is an accessible, (very) inexpensive but precise way of viewing and assessing road surfaces and other roadway features. Fortunately, the technology of AI and machine learning, when combined with a simple smartphone, can be used to solve that problem. More importantly, this technology moves the challenge of roadway assessment out of the realm of subjective opinion to the world of true data-driven decision making, and makes such a move available for even the smallest municipality or township. RoadBotics’ focus is helping municipalities, cities, states/regions and even nations monitor and maintain roadways through Artificial Intelligence in a precise, very affordable and easily accessible way.
INTRODUCTION

Roadway maintenance presents a significant issue worldwide. A 2014 report estimated that the United States would need to spend 45.2 billion dollars per year to maintain the existing infrastructure in good conditions, nearly three times the existing expenditure of 16.2 billion dollars per year (Jaffe, 2015). Pavement inspection and monitoring is a critical part of successfully maintaining roadway infrastructure that often places a significant cost and/or labor burden on public works departments and roadway managers. Current practices are split in a dichotomy: high-tech, high-cost solutions and low-tech, less capital-intensive visual surveys.

Technological advances have enabled very precise measurements of roadway distresses. For example, ARAN systems can be outfitted with numerous sensors including video, longitudinal laser profiling, transverse laser profiling, ground penetrating radar, and more (ARAN - survey subsystems, 2018). Unfortunately, systems like this require substantial capital investment. This typically means that smaller public works departments, municipalities, and even cities with substantial road networks must get inspections from contracting firms where the costs are passed on to by charging hundreds of dollars per mile. While the level of precision provided by these technologies is impressive, it is debatable whether it is critical for all applications. Furthermore, the cost burden often forces those municipalities that do decide to get these inspections into a 2-5 year inspection cycle, where either the entire network is done every few years or portions of the road network are done each year.

Often the financial burden of technological solutions deter roadway managers who then opt to perform visual surveys. This is especially true of local governments with small public works departments overseeing road networks on the order of 100 miles in length. Visual surveys, either done in-house or by contracting with engineering firms, require field crews to drive the entire network to manually make judgements and record their findings about the road condition. Furthermore, these inspections are often done at speeds below the speed limit, with crew member potentially getting out of the vehicle for closer inspection on an active roadway, potentially posing a safety hazard. These inspections are often the most practical for smaller road networks, since they require little to no upfront cost and existing resources (e.g. employees and public works vehicles) can be deployed.

Visual inspections typically follow an established rating methodology such as PASER (Walker, 2002) or Pavement Condition Index (PCI) (Pavement Condition Index 101, 2009). For asphalt surfaces, PASER involves breaking the road network into 0.5-1 miles sections in rural areas and 1-4 block sections in urban areas. Each section is given a score from 1-10 based on the presence and severity of various asphalt distresses, such as cracking, potholes, raveling, bleeding, and rutting. Similarly, PCI typically involves dividing the road network into 1 block sections in urban areas and up to 10 km (6.2 miles) sections and evaluating the presence and severity of various distresses. Once the presence and severity of each distress type is determined, they are all given a weighted contribution to a 0-100 rating. One issue with visual inspections as a whole is that while they attempt to introduce a certain level of objectivity in determining the overall score, there is a certain level of subjectivity that goes into assessing the severity and overall impact of different distresses. For example, recognizing a large pothole is something that can be done by almost anyone with little to no training, whereas distinguishing five levels of severity in raveling or rutting by visual appearance can introduce disagreement between raters.
Recent advances in computer vision and machine learning, especially with respect to deep neural networks, have empowered numerous applications. Billions of dollars are currently being invested into the sensing capabilities of autonomous vehicles as well as safety systems such as Subaru’s EyeSight technology. Similar technologies can be leveraged for automated visual assessments using video data as the basis of a low-cost sensor platform. Varadharajan et al (2014) originally proposed this idea in a paper showing the feasibility of detecting cracks in asphalt using computer vision and video data collected with smartphones. While some degree of capital investment is needed in developing these technologies, their utility can scale extremely well so that the research and development costs can be amortized across many road inspections, ultimately minimizing the costs passed on to roadway managers. In addition to advances in software, consumer demand has driven significant developments in hardware. Now, a typical smartphone contains a powerful processor, high quality camera, GPS sensor, accelerometer, gyroscope, magnetometer, and more. All of these capabilities are available for prices ranging from $200-$800. These technological advancements can be harnessed either as is or as cheaply built, application specific hardware. These hardware devices can be deployed by professional drivers, fleet vehicles (e.g. garbage trucks, package delivery vehicles, etc), or by existing public works vehicles. Overall, the combination of low-cost data acquisition platforms combined with the capability and scalability of machine learning poses an opportunity to provide efficient, affordable solutions that strike a middle ground between the precision of current high-tech solutions and lower-cost visual inspections.

2 LOW-COST SENSOR PLATFORMS

Hardware advances in the past few years have gained huge strides, primarily driven by consumer applications, such as social media. Consumer devices like a typical smartphone, dash-cameras such as those made by BlackVue, or various action cams such as the GoPro Hero line all offer relatively high quality camera sensors, accelerometers, and global positioning for only a few hundred dollars. Smart phones in particular often come with sensors such as gyroscope and magnetometer, in addition to being programmable, typically with robust software development kits (SDK). Furthermore, some models have even been equipped with stereo cameras or even built-in thermal imaging. These capabilities are packaged in a small form-factor, capable of storing an entire day’s worth of data collection on an SD card. Overall, these devices have formed the foundations of a low-cost, mobile sensor platform capable of acquiring the necessary data for road inspection. Data acquisition pipelines can be built by deploying professionally trained drivers using custom applications, free or paid crowd-sourcing, attaching hardware to existing municipally owned vehicles, or leveraging existing vehicle fleets. The latter poses an interesting opportunity, since there are garbage trucks, street sweepers, mail delivery vehicles, school buses, public transportation, and delivery trucks that already drive over nearly every street at predictable time intervals. In addition, many of the newer consumer vehicles are equipped with video and GPS capabilities that are paired with data connections. Overall, there are ample opportunities to build a low-cost sensor platform adequate for road inspection.

3 DETECTING DISTRESSES WITH DEEP LEARNING

Applications of convolutional neural networks (CNN) for computer vision have exploded recently, including systems for autonomous vehicles (e.g. Huval et al, 2015), processing satellite imagery (e. g. Chen et al, 2014), automating medical diagnoses (e. g. Esteva et al, 2017), and much more. Applications are typically classified as semantic segmentations or object identification. Semantic segmentation involves classifying each pixel of the image as one label in predetermined set of classes (e.g. sidewalk, person, pavement, etc). Object identification involves detecting the presence of a predetermined set of objects and additionally giving their location in the form of a bounding box. More recent approaches have combined the two approaches to identify the presence of objects and output their pixel level segmentation, such as with mask R-CNN (He et al, 2017).

We have developed a semantic segmentation model to identify pavement distresses. This type of model is appropriate, since in most cases the pavement distresses do not make up discrete objects (e.g. raveling, cracking). Using a custom built labeling platform (see Figure 1), trained labelers annotated selected...
distresses for video frames by “painting” over top of the distresses with a brush tool. Training the model to identify distresses then essentially becomes a process of teaching the model to recreate the “paintings” of the human labelers. In more technical terms, the annotations by human labelers are converted to binary masks (arrays of 0s and 1s) for each distress type with values of 1 denoting the presence of that particular distress for a given pixel. The model then learns to map RGB color images (MxNx3 array where MxN is the image resolution and the last dimension is the 3 colors) to a probability map for each class (MxNxP array where P is the number of classes). Figures 2 and 3 show example model predictions for various classes.

Figure 1. Annotation of distresses by painting over them using custom web-based labeling platform.
Figure 2. Selected segmentations from our deep learning model. Left column contains raw video frames. Right column contains model output where grayscale value indicates probability (i.e. black = 0%; white=100%).
4 AUTOMATED ROAD INSPECTION SYSTEM

We have developed a commercially available automated road inspection system. Using a custom developed application, we collect video and GPS data using Android smartphones mounted to the windshield with typical suction cup mounts. This setup is depicted in Figure 4. A second phone is used to navigate prepared turn-by-turn routes to ensure coverage of the desired region. Professionally trained drivers are deployed to drive the prepared routes. After collection, our application uploads video and GPS data to Google Cloud Platform (GCP), where individual frames are extracted at 3 meter intervals and indexed by time and spatial coordinates for efficient lookup.
Once the data have been uploaded and processed, scaled deployment of our deep learning model starts. Frames are processed in parallel by up to 96,000 computational workers (1000 server instances with 96 computational threads each). We typically use fewer workers (~1000), in which an entire municipality (100-200 miles) can be processed in a couple hours. Aggregate statistics are calculated on the segmentation masks per frame, which are combined into an overall 1-5 score, similar to the calculation of PCI. Example images for each rating are shown in Figure 5. After all required frames have been rated, the output enters the final stage of the processing pipeline where all of the point data (video frame with GPS coordinates, rating, and other metadata) are localized to Open Street Maps roads that have been divided into intersection-to-intersection segments. Average ratings are calculated on a per segment basis, and the final results for point and segment data are sent to our web browser-based visualization platform for delivery to the end-user. Screen-shots of network level visualization are shown in Figure 6.
Figure 5. Example images demonstrating our overall 1-5 ratings.
Figure 6. Network level display of ratings. The top image shows ratings that have been averaged on intersection-to-intersection road segments. The bottom image shows individual images that are sampled every 3 meters along the road. Users can click the points or segments to display additional information, such as to view the associated image.
5 CONCLUDING REMARKS

We have demonstrated the successful implementation of a fully automated, AI-based pavement inspection system. This system requires minimal capital investment in hardware by taking advantage of low-cost, readily available smartphones as our sensor platform. The use of cloud computing means that the system can scale efficiently. There are, however, ample opportunities to expand capabilities. For example, consider two roadway types which are not paved surfaces but are nevertheless commonplace: brick roads and gravel roads. The former are common in residential neighborhoods, as well as scenic downtowns and departments of public works do spend money to assess and maintain them. They pose a further level of challenge to AI-based assessment as they are more difficult to visually assess in the first place and visual features that could be indicative of damage in a paved surface (e.g. long cracks), may actually be normal features (e.g. the indentation separating each brick). Gravel roads, by contrast, tend to be more frequent in rural areas, but often of equal or greater importance, because gravel roads are more often than not the roads by which important capital assets are serviced. Examples include wind farms, power plants, and factories. Future efforts will focus on improving the current capabilities as well expanding them to other surface types, such as gravel and brick roads.

6 CITATIONS AND REFERENCES


ABSTRACT:

Nearly 58% of the road mileage in Illinois is known as low traffic volume roads with the traffic volume less than 400 vehicles per day which is mostly used for accessing to farm fields and other routes. Therefore, agricultural vehicles such as tractor (oversized vehicle with more than 8 axles) are heavier as compared to the normal vehicles may cause huge damage in terms of distresses to the roads. Also, local transportation agencies do not have enough resources to evaluate the progressive damage of the roads and for this reason, Illinois Department of Transportation is currently exploring the use of smartphone application to access the condition of low volume roads such as seal coated, gravel and dirt roads in terms of International Roughness Index (IRI).

In this study, low volume roads are evaluated by smartphone application known as Roadroid installed in phone mounted in sedan car for a specific distance at required speed limits from 40 to 70 kph for different sites accordingly, as well as data is compared with the truck-mounted high-speed laser profiler. Analysis shows, the IRI values from smartphone mounted on truck gives higher IRI value as compared to sedan car. The IRI variations between truck and sedan car are higher while the vehicles were driven faster.
1 INTRODUCTION

Low volume roads are the roads where the traffic count is less than 400 vehicles per day (Zegeer et al. 1994). Low to very low volume roads provide essential support in economic development by connecting farmlands, rural communities, and recreational areas to cities. About 58% of the roads in Illinois are low volume roads where the and most of the low volume roads in Illinois are seal coated roads specifically chip sealed roads. Often, full-depth asphalt concrete pavements are also constructed over compacted subgrade. Also, gravel and dirt roads provide connections to farmland from seal coated roads. Pavement distresses observed in low volume roads have few differences than conventional asphalt concrete roads due to differences in structural configuration, materials use, and type of traffic. For example, agricultural vehicles are oversize and heavier compared to the regular vehicles, and they cause excessive damage such as cracks and deformations at the outer edge of the low volume roads. Another example, a freshly prepared chip sealed road has a rougher surface, but over time the roads become smoother since the chips are embedded into underlying binders after exposed to traffic.

It is important to keep these low volume roads in shape and operable condition due to its economic importance. Improvements in rural access, in general, increase the benefits such as lower transportation costs and reduce travel time. Local transportation agencies have limited resources to measure the progressive damage of the roads. Many times, the road condition is measured by visual observation also known as “windshield survey” rather doing automated road condition survey or physical measurement of distresses. Windshield survey is subjective since the surveyor’s experience plays a major role in accessing the pavement condition. Automated road condition survey is expensive since its require sophisticated devices mounted on a truck, and the physical measurement of distresses is time-consuming and labor-intensive. For this reason, a cheap but reliable pavement conditioning measuring tool is necessary for accessing low volume roads.

2 LITERATURE REVIEW

Pavement roughness indicates the condition of roads. Smoother roads have low severity pavement distresses, and rougher roads indicate high severity pavement distresses. Pavement distresses such as fatigue cracks, transverse cracks, longitudinal cracks, rutting, and potholes make a pavement rougher. According to ASTM E867, pavement roughness can be defined as “the deviation of a surface from a true planar surface with characteristic dimensions that affect vehicle dynamics and ride quality.” As rough pavements impart vehicle accelerations, this in turn adversely affects vehicle wear, ride quality, and safety (VanDeusen 1967; Brickman et al. 1972; Abaynaka et al. 1976; Gillespie and Sayers 1981).

Roughness has been considered as the vital factor of pavement condition and rides comfort and safety for many years. It is computed by measuring the vehicle suspension motion. It is expressed by International Roughness Index (IRI) with a unit of inch/mile or m/km and is the most widely used value to summarize profile character those contribute to road characteristics. The IRI is produced using a quartet-car model and the longitudinal road profile. The IRI is the accumulation of the motion between the spring and the length of the longitudinal profile (Park 2004). The simulated suspension is accumulated and divided by the distance traveled to yield the IRI. The smaller IRI values represent a smoother road, and higher IRI values are indicative of a rougher one.

The condition of a road can be predicted on ride quality by road users if it is smooth and discomfort while driving. For future purposes, pavement condition data is required for providing pavement maintenance needs (Korchi 2004). IRI is adopted as a pavement condition reference scale for all roughness devices and equipment used for measuring road damages or defects.

Several traditional roughness measuring devices are divided into five categories:

- High-speed inertial profilers
- Response-type road roughness measuring systems
- Profilographs
- Light-weight profilers
- Walking profiler

High-speed inertial profilers are the laser survey vehicles which represents IQL-1 (Information Quality Level-1) devices measures the road profile direction and collect pavement condition data, independent of the speed at 10 to 20 m intervals. The principal components of a high-speed profiler are laser-based height sensors, accelerometer, and an accurate distance measuring system. The height sensors record the distance of pavement surface from the vehicle and accelerometers records the vertical acceleration of the sensors. The recorded results are accurate, but this device is quite expensive and not affordable for all local transportation agencies, also, it needs a skilled operator.
Response-type road roughness meters systems (RTRRMS) record the vehicle response towards pavement profile but do not record profile. They are generally affected by pavement texture and speed, therefore uses calibration equations for true values. They are referred to as IQL-3 level devices which use correlation equations for different speeds to relate IRI at 100 m+ intervals. Also, they are less expensive compared to high-speed inertial profilometers. However, it is difficult to achieve the calibration correctly because of a high degree of non-linearity in the systems which in return led to inaccuracy and inconsistency.

Profilographs are the devices used for the measurement of road roughness, irregularities, smoothness of road profiles and inspection, quality control and acceptance for pavement construction. They are the rolling wheel device consists of a rigid beam of the frame with a system of support wheels at either side and a center wheel which is connected to a strip chart recording system where the results are recorded on a graph paper from the motion of the sensing wheel. The data collected from profilograph used to calculate the IRI. Due to their massive design, they are not used frequently, and its degree of accuracy is medium.

Light-weight profilers itself represents that the profiling systems are installed in the lightweight vehicles such as golf cart or all-terrain vehicle usually used to evaluate fresh pavements. The profiling system installed in lightweight profilers is similar to the one installed in high-speed inertial laser profiler. The profilographs are obtained using different pavement sections which helps to generate profile index and bump locations which incorporate to find the other roughness related indices such as the IRI.

The walking profilometer uses an inclinometer which is fixed between the support wheels on either side to evaluate the surface profile and results collected are verified with the high-speed profiler data.

The assessment of the road roughness can be done by using the road mentioned above roughness measuring devices which collects the pavement condition data and measures the longitudinal road profiles for the selected site. Most of them are used and operated by professionals. Survey related techniques are proved accurate but are not viable and cost-prohibitive. Moreover, RTRRMs devices are coming into use during the 1960’s and with positive attributes like low cost and their ability to mount onto any vehicle. Inconsistencies due to different variations while using different vehicle types were found having different suspension systems. However, inertial laser profilers are in use nowadays commonly use acceleration measurements and are inaccurate at low speeds e.g. speed less than 15 km/hr. (Sayers, 1998).

Therefore, smartphone-based applications are being used to quantify road roughness, with the help of sensors and accelerometers from smartphones a value can be obtained and then further combined with other roughness indices to simulate IRI. AndroSensor named smartphone application was used to collect acceleration data via sensors at different driving speeds, and VIMS (Vehicle Intelligent Monitoring System) was used to calculate IRI value when driving speed of the vehicle was 20 kph or faster. The results analyzed after the data collected showed that speed has some effects on the acceleration vibration towards road roughness condition and IRI as a linear function of the magnitude of accelerated vibrations (Douangphachanh & Oneyama 2013).

There are multiple variables affecting the measurement of road roughness when using Android-based smartphone application software like type of phone, its operating system, calibration, vehicle type, speed, and surface type data all contribute to explaining the variance of IRI road segment ratings. Accelerometer based data measures the gravitational pull to determine the axis at which a device is tilted concerning the earth. Motion sensors in accelerometer are used to detect the movement of the vehicle with speed concerning time. It calculates the magnitudes of the vibration recorded from each axis. On the basis of its accuracy another research has been done to assess and evaluate the feasibility and accuracy of accelerometer data collection by using smartphone app named as Data Probe which was placed in all the nine Michigan DOT vehicles and same vehicles were used for collecting the distress data using PASER ratings for comparisons in two phases for three different years. The results showed that driving over the same road segment multiple times will increase the power of the Data Probe to more accurately predict the IRI values (Belzowski & Ekstrom 2015).

3 OBJECTIVES

The objective of this research is to use cellphone applications to measure IRI for low volume roads. Use of cellphone is considered in this research since high-speed profiler is expensive for use in low volume roads and physical distress survey is time-consuming.

The Android-based cell phone app “Roadroid” is used to measure IRI for low volume roads such as cell coated, gravel, and dirt roads. Also, a high-speed profiler is used to check the variation of app measured IRI from the profilometer. However, the profilometer is used only in two occasions since the equipment is not readily available to check for other low volume roads.
4 DATA COLLECTION PROCEDURE

High-speed laser profiler is used to access a seal coated road to check the consistency of the IRI data. Figure 1 shows the highspeed inertia profiler used in this study. The calibration distance for the profilometer is 160 m. Figure 2 shows the cell phone mounted in the truck and data collection procedure. To check the consistency of the cell phone app and the profilometer, the cell phone was mounted on the truck, and the data were collected simultaneously using both cell phone and profilometer. The truck was driven at a speed of 40 to 90 kph. Also, the cell phone was mounted in a sedan car to check the variability between truck and car.

![Figure 1](image1.png)

(a) Displacement transducer  (b) Accelerometer and laser source  (c) Output through the attached computer

Figure 1. High-speed inertial laser profiler and its parts used for measuring IRI.

The truck hosting the profilometer or the sedan car with the mounted smartphone was driven at a constant speed between the start and end of the pre-established road length. The smartphone was mounted in the sedan car, and calibration was done according to the procedure described in the smartphone application. The vehicle speed ranges from a low speed to the posted speed of the corresponding seal coated roads. The sedan car was driven in both directions (i.e., Northbound to Southbound and Southbound to Northbound), and the average IRI value was calculated for each driving speed. Also, the cell phone was mounted in a truck, and the IRI data were collected. Later the IRI data collected in the sedan car and the truck were compared to see if any differences existed between the measurements.

![Figure 2](image2.png)

(a) Smartphone mounted  (b) Coordinates calibration  (c) Data retrieved  (d) Survey data map

Figure 2. Cell phone mount in the truck and observed data

5 RESULTS

SEAL COATED ROADS

a) N Trigger Road, Edwards, Tazewell County

For N Trigger Road site visit, the data were collected by 1) cell phone mounted in the sedan car, 2) cell phone mounted in the truck and 3) profilometer attached in the truck. The speed for both sedan car and the truck varied from 40 kph to 90 kph. Roadroid recommended 40 kph as the lowest speed to collect data, and maximum of 160 kph speed was recommended. However, driving 160 kph on the seal coated roads would not be safe, and for this reason, the maximum driving speed was set to close to the posted speed. Figure 3 shows the seal coated road and the corresponding IRI values measured by the profilometer and the Roadroid app.
The IRI data measured by cell phone in the car, cell phone in the truck, and the profilometer in the truck. According to the graph, as expected, the profilometer data is consistent with an increase in speed. However, IRI values increased while the cell phone was mounted in the sedan car and the truck and the IRI values increase with an increase in vehicle speed. The increment of IRI is greater in the truck compared to the sedan car. The average IRI measured by the profilometer is 2.16 m/km, and the average IRI measured by the cell phone mounted in the car is 2.30 m/km, which is 6.89% higher than the profilometer measurement. On the other hand, the average IRI measured by the cell phone mounted in the truck is 3.75 m/km, which is 73.3% higher than the profilometer measurement.

b) N Odom Road, Benton, Franklin County

The N Odom Road is a seal coated road located at Benton with low traffic and fair conditioned pavement. Figure 4 shows the N. Odom road and corresponding IRI values.

During the N. Odom Road site visit, the data were collected by (1) cellphone mounted in the sedan car, (2) cellphone mounted in the truck. The distance at which the cellphone gathered the data was 500 m. While analyzing the data, the distance was taken as 200 m to get the accurate data for the IRI. The speed limit for both the sedan car and truck varied from 40 kph to 55 kph. This speed limit was chosen from our experience from the Tazewell County data collection which indicated that higher speed gave higher IRI values. The vehicle has driven both ways (i.e., eastbound and westbound) at a given speed and the IRI values were averaged for both directions. Figure 4 shows the average IRI data measured by cellphone in the sedan car and cell phone in the truck. According to results, the average IRI value measured by the cell phone in the car was 1.63 m/km, and in the truck, it was 2.82 m/km, which is 42.41% higher than the cell phone in the truck case.
c) Mt. Zion Road, Benton, Franklin County

The same process was followed as in the case of N. Odom Road, but we were unable to take the readings from placing the cell phone in the truck due to some technical problems in the cell phone as it was running out of battery. According to our site visit, the road was newly surfaced. Figure 5. Shows the Mt. Zion Road and the photo of the site, which shows loose aggregates exposed on the pavement surface.

![Figure 5. Seal-coated road and the measured IRI data.](image)

During Mt. Zion Road site visit, the data were collected by the cell phone mounted in the sedan car to record the IRI data at a distance fixed at 500 m. While analyzing the data, the distance was taken as 200 m to get the accurate data for the IRI. The speed limit followed was the same as for N. Odom Road. Figure 5. shows the IRI data measured by cellphone in the sedan car. Comparing N. Odom and Mt. Zion roads, the average IRI value measured in the Mt. Zion Road is higher (i.e., 3.2 m/km) than the N. Odom Road IRI value (i.e., 1.63 m/km). A higher IRI value indicates rougher pavement and a lower IRI value indicates smoother pavement. The newly surfaced Mt. Zion indicates rougher pavement. The loose aggregates on the pavement surface could influence the IRI data recorded by the cell phone. Note that the IRI values calculated in the Mt. Zion Road and N. Odom Road are more consistent when compared to the N. Trigger Rd data in Tazewell County. Lower speed gives consistent IRI reading for the seal coated roads.

d) 1200 N Road, Monticello, Piatt County

Figure 6. Shows the 1200 N road and the photo of the site and the corresponding IRI data.

![Figure 6. Seal-coated road and the measured IRI data.](image)

During the 1200 N road site visit, the data were collected by cellphone mounted in the sedan car to record the IRI values at a distance of 300 m at speed limits set for 30 to 50 kph in both directions. While analyzing the data, the
distance was taken as 200 m to get the accurate data for the IRI. The speed limit was decreased to check if more consistent data could be achieved using the cell phone.

Figure 6. shows that the IRI data measured by the cell phone in the sedan car. The IRI values were not consistent. According to Roadroid, if the speed increases it gives the better roughness value. The IRI value at a speed of 30 kph is 2.54 m/km, and at 50 kph it is 3.60 m/km, 41.51% higher than the IRI value at 30 kph.

e) N 730 E Road, Springfield, Sangamon County

For N 73 E Road site visit, the data were collected by (1) cellphone mounted in the sedan car, (2) cellphone mounted in the truck, and (3) profilometer attached in the truck. The distance between two cones was 805 m. While analyzing the cell phone data, the distance was taken as 200 m to get the accurate data for the IRI. Figure 7. shows the photo of the site and the corresponding IRI values.

Figure 7. shows the IRI data measured by cell phone in the car, cell phone in the truck, and the profilometer in the truck. Profilometer data decreases with an increase in speed, and the same result is for the car IRI value which decreases with increasing speed. However, the IRI value increases for the truck with an increase in speed at 60 kph. The increment of IRI is greater in the case of the truck compared to the sedan car. The average IRI measured by the profilometer is 4.62 m/km, and the average IRI measured by the cell phone mounted in the car is 3.85 m/km, which is 16.62% lower than the profilometer measurement. On the other hand, the average IRI measured by the cell phone mounted in the truck is 3.50 m/km, which is 8.97% lower than the profilometer measurement.

GRAVEL ROADS

a) Spring Pond Road, Benton

The similar procedure is followed for gathering the IRI data for specified distance but speed limits considered as 30 kph to 50 kph because of the road surface condition. Figures 8. and 9. show the gravel road and the picture of the site at Spring Pond Road and at Piatt County, respectively.

Figure 8. Gravel road and the measured IRI data.
b) 1300 N, Piatt County

Spring pond road shows high IRI value when the cell phone placed in the car compared to cell phone placed in the truck. The IRI value decreases for both gravel roads while speeding increases.

DIRT ROAD

600 E, Monticello, Piatt County

Figure 10. shows the picture of the dirt road surveyed and the following the results obtained from the road. IRI measurement shows a large variation between two data points.

6 CONCLUSION

This paper opens up features and relationships between data collected from smartphone application and inertial profiler. The data collected from smartphone app is considered as the potential alternative for assessing road surface condition in a cheaper and user-friendly way and can frequently be implemented. Different sites selected for seal coated, gravel and dirt roads are surveyed to obtain IRI measurements, and the data is stored and retrieved through the website and matched with the reference inertial profiler values. From the analysis, it has been found that the best IRI could be achieved by mounting the cellphone in the car drives at about 40 to 65 kph. However, the IRI data obtained from truck shows the higher values as compared to a car. Whereas, in case of dirt and gravel roads there no specific IRI scale which could show if the road is in good condition or bad, so this issue can be resolved by collecting more data on the gravel and dirt roads which further helps to develop an IRI scale of gravel and dirt roads. Also, the inconsistency should be further studied by collecting more data from seal coated roads, gravel and dirt roads in different poor, good and fair condition. The result demonstrated that cell phone could be a potential alternative since the app is cheaper and user-friendly compared to the expensive and skilled technician operated laser profiler.
REFERENCES


### PAPER TITLE

City of Phoenix Pavement Management System

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### KEYWORDS:

Asset Management, Pavement Management System

### ABSTRACT:

Cities around the world struggle with aging infrastructure. Roads are one of a City’s most visible assets. Cities are faced with the challenge of the increased need for maintenance and tight budgetary constraints. Thus, effective asset management is becoming a high priority for community leaders. This presentation will provide an overview of how the City of Phoenix, Arizona has implemented and uses its pavement management system to maintain its roads.

The City of Phoenix uses a pavement management system to monitor the condition of its roadway network and develop its maintenance program. As the fifth largest city in the United States, Phoenix faces unique challenges including a large land area, aging infrastructure, at-risk funding sources, and a ‘do more with less’ attitude. The City has taken a proactive approach in maintaining its roadways by investing in an automated data collection vehicle and a pavement management system to evaluate and maintain its nearly 5,000 mile roadway network.

The City uses an Automated Road Analyser (ARAN9000) to collect data on surface distresses and roughness. The data are processed and imported into the pavement management system. Using the pavement management system, the roads are given a Pavement Condition Index (PCI) rating and evaluated against other streets in the network. This analysis includes the type, severity, and extent of distresses to calculate the PCI. Using a life cycle cost analysis, the pavement management system develops the maintenance plan given the City’s budgets for various resurfacing treatments.

Using the pavement management system, Phoenix is able to identify streets for both preservation and rehabilitation. Recognizing the need for increased funding, Phoenix residents approved a sales tax targeted toward various transportation initiatives including street maintenance. This has allowed the City to touch more miles of roads per year than ever before, increasing the frequency at which road will be treated. The pavement management system also allows City staff to show stakeholders the effects of increasing or decreasing funding on the overall pavement condition, making it a valuable tool in talking to City leadership about future funding levels required to maintain or improve the overall pavement condition.
ABSTRACT:
The paper presents the results of a research study conducted to evaluate the long-term performance and benefit-cost of cold in-place recycling (CIR) pavements in Nevada. The long-term performance of the CIR pavements were assessed in terms of their pavement condition index (PCI) measured per ASTM D6433. The measured PCI values were used to develop performance models for CIR pavements. The costs of the CIR pavements were evaluated based on the initial costs of the CIR activity and the AC overlay or the surface treatment and the future costs of maintenance treatment applied over the life of the pavement. The benefit was determined as the area under the performance curve of the CIR pavement (i.e. PCI versus years). The benefit-cost ratio was calculated as the benefit over the present worth cost for each CIR pavement times 100. The analysis of the long-term performance and benefit-cost ratios led to the following conclusions: a) majority of CIR pavements constructed in Nevada during the period of 2000 – 2015 have been performing at the level between very good and excellent, b) majority of CIR pavements constructed in Nevada during the period of 2000 – 2015 have shown benefit-cost ratios in the range of medium to high, and c) there is an inverse relationship between the thickness of the CIR layer and AC overlay and benefit-cost ratio indicating that the structural pavement design method is not capturing the full behavior of the CIR layer.
1 INTRODUCTION

The Nevada Department of Transportation (NDOT) has constructed over 2500 centerline kilometers of Cold In-Place Recycling (CIR) pavements in the past 15 years. Depending on the traffic volume of the road, NDOT has been constructing two types of CIR projects. For high volume roads, typically a 50 - 75 mm CIR layer has been applied, followed by the required structural asphalt concrete (AC) overlay (38 – 100 mm) and an 19 – 25 mm open grade friction course (OGFC) or a surface treatment as the wearing surface. For low volume roads, a 50 – 75 mm CIR layer has been commonly applied, followed by a surface treatment such as chip seal, double chip seal, or microsurfacing as the wearing course (Piratheepan et al 2014). This research assessed the long-term performance and conducted benefit-cost analysis of CIR pavements throughout Nevada constructed over the period of 2000 – 2015.

2 LONG-TERM PERFORMANCE OF CIR PAVEMENTS IN NEVADA

Long-term performance of CIR pavements throughout Nevada was evaluated based on condition surveys obtained from NDOT’s Pavement Management System (PMS) conducted on 2-years cycle. The CIR projects were divided into two categories; projects with CIR layer and AC overlay, and projects with CIR layer and surface treatment. The study identified 94 CIR pavements in Nevada from NDOT database constructed during the period of 2000 - 2015; 63 CIR pavements with AC overlays, and 31 CIR pavements with surface treatments.

Tables 1 and 2 summarize the CIR pavements with AC overlay and surface treatment selected for the long-term performance and benefit-cost analysis, respectively. Projects with less than 10 years of condition survey information were excluded from the performance analysis. Based on this exception, the 94 pavements represent all the CIR pavements that were constructed by the Nevada DOT over the period of 2000 – 2015 with performance data equal or more than 10 years.

The pavement condition index (PCI) is a pavement quality indicator calculated based on the severity and extent of distresses identified on the pavement surface following the procedure identified in ASTM D6433 (ASTM 2018). PCI ranges from 0 to 100, with 100 being the best possible condition of the pavement and 0 the worst. Tables 3 and 4 summarize the PCI values of the selected CIR pavements with AC overlay and with surface treatment used for performance modeling and benefit-cost analysis, respectively.

The first step of the benefit-cost analysis is to develop performance models for the two types of CIR pavements in order to determine the benefits realized by each activity. The PCI data calculated from NDOT condition surveys were used to develop performance prediction models using polynomial regression, as shown in Figures 1 and 2 for CIR pavements with AC overlay and CIR pavements with surface treatment, respectively (Ortiz 2018). Table 5 summarizes the performance models along with the applicable ranges of models parameters. It is highly recommended not to use the models to predict the performance of CIR pavements with parameters outside the ranges specified for each model.

The PCI data shown in Figures 1 and 2 indicate that most of the projects are in excellent or very good condition, until the age of 15 years for CIR pavements with AC overlay and 12 years for CIR pavements with surface treatment. Using the performance prediction models, the PCI was estimated up to the 15th year for CIR pavements with AC overlay having less than 15 years of actual performance data and until the 12th year for CIR pavements with surface treatment having less than 12 years data. The estimated PCI values are shown in red and underlined in Tables 3 and 4. The predicted PCI values were adjusted using a correction factor for each project, calculated as the difference between the last measured PCI value and the respective predicted PCI value at the same year (Ortiz 2018).
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Figure 1. Performance model for CIR pavements with AC overlay.

y = 0.0008x^6 - 0.0365x^5 + 0.6524x^4 - 5.6302x^3 + 24.15x^2 - 47.304x + 127.88
R² = 0.7745

Figure 2. Performance model for CIR pavements with surface treatment.

y = -0.0031x^6 + 0.12x^5 - 1.7976x^4 + 13.091x^3 - 47.225x^2 + 75.522x + 51.807
R² = 0.6185
Table 5. Performance models for CIR pavements in Nevada

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<th>Range of AC Overlay Thickness (in)</th>
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<td>PCI = 0.0008 * Age^6 - 0.0365 * Age^5 + 0.6524 * Age^4 - 5.6302 * Age^3 + 24.15 * Age^2 - 47.304 * Age + 127.88</td>
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3 BENEFIT-COST ANALYSIS

A benefit-cost analysis was used to determine the relative cost-effectiveness of the different CIR projects throughout Nevada and identify key factors that led to more effective pavement treatment. The benefit-cost ratio is defined as the ratio of the benefit offered by the CIR divided by the cost of the project. In this study, the cost of the project was defined as the initial construction cost of the CIR application, the cost of overlay or surface treatment application, and the cost of any maintenance treatment applied during the analysis period. The Present Worth (PW) of all future costs was calculated and used for the benefit-cost analysis. Projects with insufficient cost information were eliminated from the benefit-cost analysis which resulted in 29 and 25 projects for CIR with AC overlay and with surface treatment, respectively.

The costs of the CIR, overlay, and surface treatment were determined using bid information of various projects obtained from NDOT. As an example, Table 6 summarizes the cost of the CIR application and the cost of the AC overlay application for contract 3013. These costs were divided by the number of lanes and the number of kilometers of each project to determine cost per lane-km. In this example, project 3013 had two lanes and 31.36 km, therefore, the CIR cost per lane-km was calculated as follows:

\[ \text{CIR-Cost} = \frac{938250}{2(\text{lanes}) \times 31.36 \text{ (km)}} = 14960/\text{lane-km} \]

and

\[ \text{Overlay-Cost} = \frac{1814734}{2(\text{lanes}) \times 31.36 \text{ (km)}} = 29934/\text{lane-km} \]

The costs of CIR and AC overlay are considered as initial costs for project 3013. The costs of future maintenance treatments will be applied at their corresponding year of application and used in the PW analysis to determine the total cost of project 3013 over the 15-years analysis period.
Table 6. Costs for a CIR pavement with AC overlay on contract 3013 at year of construction

<table>
<thead>
<tr>
<th>Layer</th>
<th>Item</th>
<th>Unit</th>
<th>Unit Price ($)</th>
<th>Amount</th>
<th>Cost ($)</th>
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<td>Lime</td>
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<td>125</td>
<td>814</td>
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<td></td>
<td>Recycled Bituminous Surface (75mm depth)</td>
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<td>2</td>
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<td></td>
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<td>250</td>
<td>814</td>
<td>203500</td>
</tr>
<tr>
<td></td>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>938250</strong></td>
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<tr>
<td>Overlay</td>
<td>Plantmix Bituminous Surface Aggregate (Type 2)</td>
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<td>59050</td>
<td>1299100</td>
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<tr>
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<td>Milled Rumble Strips</td>
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<td></td>
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<td><strong>1814735</strong></td>
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</table>

The next step in the cost analysis was to identify the future maintenance treatments that were applied on every CIR project. Maintenance treatments were applied on most CIR projects during their service life. Table 7 summarizes the identified maintenance treatments for the selected CIR projects. On average, the first maintenance treatment for CIR pavements with AC overlay was applied close to the 8th year after construction, and the second maintenance treatment was applied around the 11th year after construction. For CIR pavements with surface treatment, on average, the first maintenance treatment was applied close to the 4th year after construction and the second maintenance treatment was applied around the 8th year after construction. The most used maintenance treatments were chip seal and flush seal. This information was obtained from NDOT’s pavement maintenance management system and are summarized in Table 7.

Since the available bid information of each project included only the initial construction costs, future maintenance treatments costs were calculated as the average cost of the treatment, using bid information of several projects applying that specific treatment. Once the cost per lane-km of the CIR, AC overlay, surface treatment, and future maintenance treatments were determined, the PW at the year of construction was calculated for each project, as summarized in Tables 8 and 9 for CIR with overlay and CIR with surface treatment, respectively. Average costs of CIR and surface treatment were assumed for projects without bid information as shown in red/underlined in Table 9.

The benefit is defined as the area under the performance curve of the pavement during the analysis period. The benefit was calculated using areas of triangles and rectangles under the PCI curves. Figure 3 shows the PCI curve for contract 3013 over an analysis period of 15 years as predicted by the performance model for CIR with AC overlay presented in Figure 1 and Table 5. The total area under the PCI curve in Figure 3 was subdivided into triangles and rectangles and the overall benefit was calculated as follows:

\[
Benefit_{3013} = 100 \cdot 4 + \frac{(100 - 95) \cdot 2}{2} + 95 \cdot 2 + \frac{(95 - 94) \cdot 2}{2} + 94 \cdot 2 + [94 \cdot 5] + \frac{(94 - 93) \cdot 1}{2} + 93 \cdot 1 + \frac{(98 - 93) \cdot 1}{2} + 93 \cdot 1
\]

\[
Benefit_{3013} = 400 + 195 + 189 + 470 + 93.5 + 95.5 = 1443
\]

Table 10 summarizes the benefit-cost ratios of the CIR pavements with AC overlay selected for this study over analysis period of 15 years. Some factors that may influence the benefit cost ratio were identified as; CIR thickness, AC overlay thickness, asphalt emulsion type, and average annual daily traffic (AADT).
Table 7. Maintenance treatments applied to CIR Pavements

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<tr>
<th>Contract</th>
<th>Treatment 1</th>
<th>Treatment 2</th>
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<td>Year</td>
<td>Type</td>
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<tr>
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<td>Flush seal</td>
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<td>Chip seal &amp; flush seal</td>
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<td>2010</td>
<td>Flush seal</td>
</tr>
<tr>
<td>3139</td>
<td>2010</td>
<td>Chip seal &amp; flush seal</td>
</tr>
<tr>
<td>3143A</td>
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<td>Chip seal &amp; flush seal</td>
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Table 8. Cost per lane-km of CIR pavements with AC overlay and Present Worth at year of construction

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<th>Contract ID</th>
<th>CIR Cost ($/lane-km)</th>
<th>Overlay Cost ($/lane-km)</th>
<th>Maintenance 1 Cost ($/lane-km)</th>
<th>Maintenance 2 Cost ($/lane-km)</th>
<th>PW at year of Construction ($/lane-km)</th>
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Table 9. Cost per lane-km of CIR pavements with surface treatment and Present Worth at year of construction

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<th>Contract ID</th>
<th>CIR Cost ($/lane-km)</th>
<th>Surface Treatment Cost ($/lane-km)</th>
<th>Maintenance 1 Cost ($/lane-km)</th>
<th>Maintenance 2 Cost ($/lane-km)</th>
<th>PW at year of Construction ($/lane-km)</th>
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Figure 3. Performance curve for contract 3013 predicted using the performance model for CIR with AC overlay.

Figure 4 shows the benefit-cost comparison of the selected CIR pavements with AC overlay over analysis period of 15 years. Benefit-costs were divided into three levels; low for benefit cost ratio less than 1.5%, medium for benefit cost ratio between 1.5 and 3%, and high for benefit cost ratio higher than 3%. From the estimated benefit cost data, 31% of the projects had a high benefit cost ratio, 65% of the projects had medium benefit cost ratio, and 14% of the projects had low benefit cost ratio.

Projects with lower AADT tend to have higher benefit-cost ratio. The average AADT of CIR with overlay projects with high benefit-cost ratio was 706 vehicles, while for projects with medium benefit-cost ratio the average AADT was 2348 vehicles, and for projects with low-benefit cost ratio the average AADT was 5518 vehicles.

On average, the CIR thickness of projects with high benefit-cost ratio was 65mm, while for medium benefit-cost ratio the average CIR thickness was 70mm, and for low benefit-cost ratio the average CIR thickness was 75mm. The inverse relationship between benefit-cost ratio and thickness of CIR and AC overlay is caused by the inter-dependency between traffic level and layers thickness.

The analysis indicates that the influence of traffic on the benefit-cost is highly significant and could not be balanced by the recommended thickness of the CIR and AC overlay. This may indicate that the structural design method used for CIR pavements does not properly take into consideration the traffic level and the properties of the CIR layer. Since the asphalt emulsion used for all the CIR pavements with AC overlay was CMS-2s, the impact of asphalt emulsion type on the benefit-cost could not be investigated.
<table>
<thead>
<tr>
<th>Contract ID</th>
<th>PW at Construction Year ($/lane-km)</th>
<th>Predicted Benefit over 15 years</th>
<th>Predicted Benefit-Cost (B/C x 100) over 15 years (%)</th>
<th>CIR Thickness (mm)</th>
<th>Overlay Thickness (mm)</th>
<th>Emulsion Type</th>
<th>AADT</th>
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The same analysis was conducted for the CIR pavements with surface treatment. Table 11 summarizes the benefit-cost ratios of the CIR pavements with surface treatment over an 8-years analysis period. Some factors that may influence the benefit cost ratio were identified as; CIR thickness, asphalt emulsion type, and average annual daily traffic (AADT).

Figure 5 shows the benefit-cost comparison of the selected CIR pavements with surface treatment over analysis period of 8 years. Benefit-cost ratios were divided into three levels; low for benefit-cost ratio less than 1.25%, medium for benefit-cost ratio between 1.25 and 2.25%, and high for benefit-cost ratio higher than 2.25%. From the estimated benefit-cost data, 40% of the projects had a high benefit-cost ratio, 36% of the projects had medium benefit-cost ratio, and 24% of the projects had low benefit-cost ratio.

As in the case of CIR pavements with AC overlay, CIR pavements with surface treatment with lower AADT tend to have higher benefit-cost ratio. The average AADT of CIR pavements with surface treatment with high benefit-cost ratio was 199 vehicles, while for projects with medium benefit-cost ratio the average AADT was 516 vehicles, and for projects with low benefit-cost ratio the average AADT was 1313 vehicles.

On average, the CIR thickness of projects with high benefit-cost ratio was 50mm, while for medium and low benefit-cost ratio the average CIR thickness was 75mm. The type of asphalt emulsion used did not have a considerable impact on the benefit cost ratio. The analysis indicates that the influence of traffic on the benefit-cost is highly significant and could not be balanced by the recommended thickness of the CIR.
Table 11. Benefit-cost analysis for CIR pavements with AC overlay over a 15-years analysis period

<table>
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<th>Contract ID</th>
<th>PW at Construction Year ($/lane-km)</th>
<th>Predicted Benefit over 8 years</th>
<th>Predicted Benefit-Cost (B/C x 100) over 8 years (%)</th>
<th>CIR Thickness (mm)</th>
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4 SUMMARY AND CONCLUSIONS

This research effort evaluated the long-term performance and the benefit-cost of CIR pavements with AC overlays and with surface treatments that have been constructed by the Nevada DOT over the period of 2000 – 2015. The long-term performance was evaluated in terms of the PCI values measured over the service life of the pavement and performance models were developed for the two types of CIR pavements. The benefit of each CIR pavement was determined as the area under its corresponding performance curve over the analysis period. A total of 94 CIR pavements were included in the evaluation. Based on the analysis of long-term performance and benefit-cost ratio of CIR pavements in Nevada, the following conclusions can be made:

- The majority of CIR pavements constructed in Nevada during the period of 2000 – 2015 have been performing at the level between very good and excellent based on the ASTM-PCI scale.
- The majority of CIR pavements constructed in Nevada during the period of 2000 – 2015 have shown benefit-cost ratios in the range of medium to high.
- There is an inverse relationship between the thickness of the CIR layer and AC overlay and benefit-cost ratio indicating that the structural pavement design method is not capturing the full behavior of the CIR layer. Based on this observation, this research effort has been expanded to determine the engineering and performance properties of CIR mixtures and to incorporate these properties into the structural pavement design method.
Figure 5. Benefit-cost ratios for CIR pavements with surface treatment over an 8-years analysis period.

5 REFERENCES


KEYWORDS:
Include up to 5 keywords
Comfort Pedestrian Evaluation Network Wheelchair

ABSTRACT:
Walking and cycling are green transport modes which should be given more attention in future. The quality of footways is not always very good. Maintenance planning and budgeting are mostly based on ad hoc repairs. The problem is the lack of a complete evaluation method. We have developed an evaluation method for footways. Our approach is based on a network analysis with a wheelchair, which is the most “sensitive” type of footway user. This wheelchair is equipped with sensors and a skid resistance measuring device. Three parameters (comfort/longitudinal unevenness in the direction of travel, gradient in the direction of travel, and crossfall) can be measured in one run, and a skid resistance measurement is made in each section of the network.
We would like to present this measuring equipment with results maintained until now. Furthermore we would like to promote this approach and show the good relation between the objective measuring device and the subjective rating of the testpanel.
Footway Evaluation

1. INTRODUCTION

Since the beginning of the 21st century, accessibility of public spaces has become a growing priority for both regional and municipal road authorities. The needs of road users are choice criteria that are of great importance in the design of a new road layout.

Pedestrian accessibility depends on, among other things, the availability of high quality pavements that meet the needs of both disabled and disabled pedestrians: flatness, stability, skid resistance, obstacle, water drainage, visibility and cleanliness. Pavements meeting these needs offer high user quality, which is more likely to move in the public space concerned.

At the national and international level, there is currently no objective tool to determine this quality of pedestrian pavements continuously, quickly and cost-effectively. The concept is all too often approached subjectively, on the basis of the feeling that we get as a traffic participant in a public space. That is why the Belgian Road Research Center decided in 2015 to develop a measuring instrument to assess pavements on three criteria that are fundamental for pedestrians: flatness (comfort), skid resistance (resistance to slipping) and slope (transverse and longitudinal).

2. Evaluation criteria

Relevant technical parameters to be collected for footway evaluation are the following:

- footway width;
- longitudinal unevenness / comfort;
- crossfall / gradient;
- skid resistance / coefficient of friction;

2.1 Footway width

Different users need different footway widths. In this concept we use a wheelchair. We assume that people with impaired mobility are the most “sensitive” users of a footway. If people with a wheelchair, baby car or other vehicle can travel in an acceptable way, the criterion is met.

2.2 Longitudinal unevenness / comfort

The feeling of comfort on a footway varies widely with the type of user. In the concept proposed in this article, we will measure comfort with a wheelchair equipped with an accelerometer. This evaluation is objective and stringent, because a wheelchair user suffers the most from lack of comfort on a footway.

2.3 Crossfall / gradient

Crossfall should normally be lower than 2 %. This parameter will be measured continuously in the concept proposed below.
3. Concept

3.1 Basic concept (V0)

After analysing the available tools and the way inspections are made in other countries, we found that no existing device can measure all the relevant characteristics at an affordable price. That is why we are suggesting the concept described below. (This measurement method remains to be refined and the measuring equipment still needs to be developed.)

The basic component of the method is a wheelchair. We propose to convert it into a measuring vehicle. The chassis of the wheelchair as well as the wheels and the load (weight in the wheelchair) will be determined experimentally, as a first step. It seems interesting to choose the chassis and wheels so as to minimize suspension and damping, with a view to obtaining a highly sensitive measuring device.

![Measurement setup](Fig.1. Source: BRRC)

A smartphone is fixed onto the chassis and the following parameters can be measured:
- time;
- GPS;
- orientation with respect to the north (compass);
- speed;
- crossfall / gradient;
- longitudinal unevenness / comfort (with accelerometer).

In 2016, a first prototype (V0) was developed by the CRR. This was materialized by a wheelchair on which smartphones containing a GPS and an accelerometer had been fixed (equipment integrated by default in all smartphones). This equipment provided figures concerning the comfort of the coating (rating out of 10). This comfort is evaluated via the accelerometer which will measure the vertical accelerations generated by the surface of the coating on the wheelchair and therefore in fine on the user. In order to measure also the adhesion of the coating, a complementary tool available to the BRRC has been used: the Portable Friction Tester (PFT - more information available on www.brrc.be/fr/article/f1301_06 and www.vti.se/en/Publications/Publication/road-marking-friction_669533).

At the end of 2016, surveys were carried out with this prototype and the PFT tool on eleven "test" sites located in the city center of Brussels, each with a different coating: two asphalt, two concrete and seven natural stone.

In April 2017 and in collaboration with Brussels Mobility, the adherence and flatness of these eleven sites were then evaluated by different users during a morning organized by the CRR. The objective of this approach was to verify if the
results obtained by the measuring equipment, reflected the feeling of pedestrians. In total, twenty-eight participants took part in this field exercise: eighteen valid pedestrians and ten people with reduced mobility (people in wheelchairs, people walking with difficulty, people with visual impairments). A test sheet was completed by each participant and for each site.

At the end of the analysis and as illustrated in the graph of FIG. 5, a classification of the sites according to the comfort of the coating perceived by the users was carried out. The classification obtained from the subjective data collected by the twenty-eight users (light blue on the graph), compared to that obtained with the encrypted data collected by the prototype (dark blue), shows that a correlation exists on this notion of comfort.
Encouraged by this encouraging observation, the BRRC decided during the summer of 2017 to develop a second prototype (V1) with several objectives:

- also measure the transverse and longitudinal slopes of the pedestrian zones;
- measuring the speed of movement to eliminate the disturbances generated by different survey speeds;
- remove as much as possible the possible disturbances related to the quality of the accelerometer used (variable depending on the smartphone);
- develop a single system with which all the components communicate;
- automatically centralize all the data collected in a single database.

Results and perspectives
Presented in preview on the stand of the BRRC at the Belgian Road Congress in Brussels at the beginning of October 2017, this equipment allows to measure continuously and in a "geolocated" way the comfort and the longitudinal and transversal slopes of the sidewalks and other pedestrian spaces. The measurement of adhesion can currently not be performed by this equipment and always requires the use of additional equipment (PFT tool for continuous measurement (tests in progress), SRT pendulum (Skid Resistance Tester for point measurement). This second prototype and the sensors that compose it are shown in Figure 6.

![Figure 6: Equipment and sensors of the second prototype of the BRRC.](source: BRRC)
Currently, the CRR is finalizing equipment calibration (survey speed, total equipment mass) based on the results collected by pedestrian users as a reference value in terms of comfort.

As soon as this calibration step is complete, the development of this equipment will continue with the completion of new comfort and adhesion measurements of pedestrian coatings. At the same time, users will also be invited to evaluate these different coatings. A comparison similar to that made with the previous prototype will be proposed to confirm the correlation already observed and thus validate the tool.

At the end of this development, this functional equipment should then integrate the family of roadside inspection equipment available to the BRRC. It can then be used to respond to external technical assistance specifically oriented to the quality of use of pedestrian coatings.

The road administrator will be able to categorize different pedestrian coatings based on comfort and adhesion measurements. The following figure shows a decision grid that can be used to trigger an intervention. In the green area (VG Very Good) we have a pedestrian coating with very good scores on comfort and adhesion. In the red zones we have at least one problem (adhesion or comfort). Based on this we can choose a substantiated measure.

![Evaluation of the quality of use of pedestrian coatings](Fig.5. Source: BRRC)

Figure 1 - Prototype 1 equipped with two smartphones (with optional a camera for taking pictures)
Figure 2 - Users' assessment of the comfort and adhesion of different coatings
Figure 3 - Classification of the different sites tested by the users and by the prototype 1
Figure 4 - Tool for measuring the quality of use of pedestrian coatings under development (prototype 2)
Figure 5 - Evaluation of the quality of use of pedestrian coatings

5. What still remains to be done

The concept presented above has been tested in actual practice. We will explore different strategies to get this equipment on the market.
- the wheelchair community has shown interest and we will explore possisibilities how to encourage them to measure their network;
- Explore possibilities to use applications (app) on smartphone / tablet and correlate wheelchair users with the prototype;
- Establish a measurement method so we obtain repeatable and reproducible measurements.
6. Conclusion

Walking and cycling are green transport modes which will get more attention in future. The quality of footways is not always very good. In most cities, road managers have difficulties in estimating the quality level of their footway network. Maintenance planning and budgeting are mostly based on ad hoc repairs. With this complete evaluation method administrators will be able to manage this asset in a more efficient way and with a well determined approach. After this inspection, the administrator will have a good overview of the quality of his footway network. He will easily find those footways who need a maintenance. On the other hand, experience will be gained for different types of pavements for footways. With this experience, the administrator will be able to decide which type of pavement is suitable for which footway.
Verification of the Effect of Simple Crack Investigation Method

**PAPER TITLE**

**TRACK**
1.1 Pavement & Bridge Management Systems (Research and Implementations)

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<td>Oriental Consultants Co., Ltd,</td>
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</table>

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**KEYWORDS:**
Asset Management, Pavement, Road surface property measurement, Crack rate

**ABSTRACT:**
Crack rate is one of the most important factor for maintaining and managing roads condition in Japan. However, existing investigation methods using road surface property measuring vehicles is expensive and difficult to measure on narrow roads because of vehicles size.

Based on the previous survey results, crack rate is extremely high in model Area, S-city, Japan.

Therefore, surveys were conducted using a simple method which using general vehicle with video camera and crack rates are automatically calculated from the photographed image. It was confirmed that simple methods accuracy was secured by comparing the result of existing method and simple method.
Verification of the effect of simple crack investigation method

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1 INTRODUCTION
Management of cracking rate, rutting amount and flatness are required as the indicator of road surface properties by the pavement inspection procedure (Table1; Damage Evaluation of pavement, Ministry of Land, Infrastructure, Transport and Tourism, 2016).

Existing method is conducting survey using road surface property measuring vehicles (Figure 1), but a certain road width is necessary for measurement. Therefore, it is unsuitable for residential roads with narrow road width. Moreover, it is not realistic to measure all the living roads by the road surface property measuring cars with high running costs, due to budget and human resource shortage.

In this research, we investigated the new technology which is at less than the cost of existing method and can survey with narrow road width as well.

Table 1. Damage Evaluation of pavement

<table>
<thead>
<tr>
<th>Soundness Classification</th>
<th>Crack rate</th>
<th>Rutting amount</th>
<th>Flatness</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>&lt; 20 %</td>
<td>&lt; 20 mm</td>
<td>&lt; 3 mm/m</td>
</tr>
<tr>
<td>II</td>
<td>20 %~40 %</td>
<td>20 mm~40 mm</td>
<td>3 mm/m~8 mm/m</td>
</tr>
<tr>
<td>III</td>
<td>&gt; 40 %</td>
<td>&gt; 40 mm</td>
<td>&gt; 8 mm/m</td>
</tr>
</tbody>
</table>

Figure 1. road surface property measuring
Model area, S-city, in Japan manages about 871.7 km of road. Many of the roads were laid in 1940 (High Economic Growth Period in Japan), and the aging of pavement is proceeding. It is expected that a lot of expenses will be required for repairing the road. In order to reduce repair costs, leveling and accountability to citizens, it is necessary to transition from ex-post management to preventive management.

There are many pavements with high crack rate and narrow roads that cannot pass through the road surface property measuring vehicles.

Figure 2 shows the results of the road surface property measurement of the past year. The road in S-city shows that damage due to cracks is remarkable. There are many places where the crack rate exceeds 40% which is the judgment line of repair. On the other hand, the rutting amount is in the range of approximately 20 mm (maximum 30 mm), and the damage is minor. Therefore, it is considered that the crack rate greatly affects the soundness of the road surface in S-city.

Figure 2. The road surface property measurement

3 SIMPLE CRACK MEASUREMENT

Simple crack measurement (Figure 3) will be carried out and measurement by the road surface property measurement vehicle will be not carried out. In the calculation method of the crack rate, the cracked portion is discriminated from the photographed image of the video camera installed in the rear part of the passenger car body and calculated. From the existing research, it has been cleared that the simple method has the same tendency as the known measurement result (Kurata (2015)). The method of calculating the crack rate is in conformity with the pavement survey and test method handbook (Japan Road Association, June 2007).

The difference from existing measuring vehicles is shown in the Table 2. In the simplified method, only the crack rate can be measured among the three elements of the road surface property investigation. Also, in the simplified method, the traveling speed limit is 40 km / h, which is inferior to the road surface measuring vehicle that can be measured at 100 km / h. However, measurement on the highway is not necessary for the living road, and survey of the road surface may be sufficient only by examination of the crack rate.

When compared in terms of economics, simple measurement methods are overwhelmingly cheaper than existing methods. In addition, it is unnecessary to arrange measuring vehicles and hiring drivers or technicians, so flexible investigation can be carried out.

Existing road surface property measuring vehicles and simple measurement method of crack rate have different active fields, and by using both, it is possible to carry out more efficient road surface property measurement.
Table 2. The difference between simple method and existing method

<table>
<thead>
<tr>
<th></th>
<th>Existing Method</th>
<th>Simple Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>The size of the vehicle</td>
<td>6.2m x 2.2m x 2.8m</td>
<td>4.5m x 1.6m x 1.5m (depend on vehicle size)</td>
</tr>
<tr>
<td>Cost</td>
<td>500 USD / km</td>
<td>200 USD / km</td>
</tr>
<tr>
<td>Measurement item</td>
<td>Crack rate, Rutting amount and Flatness</td>
<td>Crack rate</td>
</tr>
<tr>
<td>Vehicle speed</td>
<td>Up to 100km/h</td>
<td>Up to 40km/h</td>
</tr>
<tr>
<td>Arrangement</td>
<td>• Road surface property measuring cars • Engineers</td>
<td>• Passenger Car • Video Camera</td>
</tr>
</tbody>
</table>
4 MEASUREMENT RESULTS

4.1 MEASUREMENT RESULTS BY SIMPLE METHOD

We conducted a crack rate survey by Simple method for 32 routes in S-city.

The results of the simple method are shown in the figure 4 and 5. In 8 routes out of 32 routes, the crack rate exceeded 40% on half of route.

Figure 4. Measurement result by simple method

Figure 5. Captured Image on route O

It is necessary to check the consistency of the results of the Simple method. We examined the route R and O which have survey results of the existing method in 2013.

Since Simple method and Existing Method differ in the survey year, comparison of the values themselves is not appropriate. Therefore, we compare the results of both surveys in consideration of aged deterioration.

Comparison crack rate between simple method (2017) and existing method (2013) is shown in figure 2 and 3. The cracks of the pavement deteriorated from 2013 to 2017, and there was no contradiction from the viewpoint of pavement deterioration.

The increase in crack rate of Route N is approximately 15 to 35%, Route O is approximately 20 to 30%. Both routes showed almost similar deterioration.

![Comparison of crack rate between simple method (2018) and existing method (2015) on Route N and O](image)

4.2 COMPARISON CRACK RATE BETWEEN SIMPLE METHOD AND PREDICTION CRACK RATE

Calculation formula of prediction crack rate is shown in the table 3. Based on the survey results of the existing method and the degradation prediction formula, the crack rate of 2018 was predicted. The deterioration prediction formula was derived from aged deterioration of the same kind of pavement in the surrounding area.

Predicted values of crack rate and measured values of Simple method in table 4 and 5. Comparison crack rate between simple method and prediction crack rate is shown in figure 3and 4. Crack rates of simple method and prediction were nearly close. Since high correlation was also confirmed, the result of simple method is considered to be worthy of practical use.

<table>
<thead>
<tr>
<th>Traffic Volume / day (Large-sized vehicle)</th>
<th>Calculation formula of prediction crack rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;250</td>
<td>( C_{i+1} = 1.0810C_i + 1.4438 )</td>
</tr>
<tr>
<td>100–250</td>
<td>( C_{i+1} = 1.0455C_i + 1.9872 )</td>
</tr>
<tr>
<td>250–1000</td>
<td>( C_{i+1} = 1.1553C_i + 0.7372 )</td>
</tr>
</tbody>
</table>

\( C_i \) : Crack rate on i year

\( C_{i+1} \) : Crack rate on i +1 year
Table 4. Predicted values of crack rate and measured values of Simple method on Route N

<table>
<thead>
<tr>
<th></th>
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<td>8.3</td>
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</table>

Table 5. Predicted values of crack rate and measured values of Simple method on Route O

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</tbody>
</table>

Figure 7. Comparison crack rate between simple method and prediction crack rate on Route N and O
4.3 CONSIDERATION OF PREDICTION OF CRACK RATE

Future prediction and results of simple method were generally in agreement, but strictly a slight difference was recognized. The variation between the predicted value and the measured value was small on the N route with a low crack rate. On the other hand, the variation was large in O route with high crack rate. As a reason, the deterioration prediction formula used in this study, the deterioration rate tends to increase when the initial value is high, possibly not conforming to reality. (Figure 8). In order to improve the accuracy of verification, it is necessary to improve the accuracy of the deterioration prediction equation by accumulating data.

![Figure 8. Verification of deterioration speed on Route O](image)

5 CONCLUSION

There was no trend difference in the results of the simple method and the existing method of road surface property measuring.

Moreover, there is no big difference between the result of the simple and the deterioration prediction calculated from the existing data. Therefore, it is considered that the simplified method is worth the practical use of crack investigation. In the future, it is necessary to simultaneously measure the same range by the simple and the existing method, and confirm the difference of the result.

Existing road surface property measuring vehicles and simple measurement method of crack rate have different active fields, and by using both, it is possible to carry out more efficient road surface property measurement. In addition, it is necessary to study simplified new method of the flatness and rutting, and to utilize low cost road surface property survey technology for road management of national and local governments.

6 REFERENCES

Ueta tomotaka(2018). Proposal of road maintenance and management method on road with high crack rate. IRF Global R2T Conference 2018


PAPER TITLE | GUIDELINES FOR COST EFFECTIVE FLEET ASSET REPLACEMENT  
---|---  
TRACK | ASSET MANAGEMENT  
AUTHOR (Capitalize Family Name) | POSITION | ORGANIZATION | COUNTRY  
Golam SARWAR | Senior Consultant | AgileAssets Inc. | USA  
CO-AUTHOR(S) (Capitalize Family Name) | POSITION | ORGANIZATION | COUNTRY  
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KEYWORDS:  
Fleet Replacement Analysis, Replacement Guidelines, Life Cycle Cost Analysis  
  
ABSTRACT:  
Replacement analyses are aimed at measuring the extent to which an equipment is within or exceeds certain established replacement criteria. AASHTO subcommittee on Maintenance has identified replacement analysis as one of the four key fleet system performance measures for State Department of Transportation for cost effective and safe fleet asset management operations. Timely and planned replacement is important because it affects vehicle safety, reliability, maintenance and operating costs.  

Each organization must develop and use replacement guidelines to track the health of the fleet system and plan for replacement spending needs. Replacement thresholds are expressed in terms of organization preferred criteria based on equipment age, usage, down time, repair costs, life cycle costs, and similar factors. The guidelines should initiate the need to replace individual vehicle/equipment for which the established replacement criteria exceed the set replacement threshold. The best practice for an organization is to develop replacement guidelines based on empirical analysis of the relationship between equipment age, useful life, cumulative usage, downtime, LTD repair cost and life cycle operating cost.  

This paper describes industry best practices in establishing criteria and threshold for fleet replacement and presents a replacement analysis tool to develop replacement guidelines. This paper also describes an example implementation of fleet replacement analysis for a US Department of Transportation using AgileAssets Inc. Fleet Management software module. This powerful analysis tool will help the decision makers identify the short and long terms need of replacement budgets, maximize residual value of existing fleet asset and avoid unexpected fleet retention.
INTRODUCTION

Fleet asset is an integral part of the highway operations and maintenance. A healthy and efficient fleet management system is essential not only to reduce fleet operating cost but also to increases the reliability of performing maintenance activities on highways in a timely manner. Like other assets, fleet also has a decay cycle which cannot be avoided. Highway agencies spend significant amount of money every year to operate and maintain their fleet along with invest immense capital investments to replace fleet assets (NCHRP 13-04 report 2018). When a fleet asset is in service for more than it’s expected useful life or recommended mileage, the probability of breakdown increases significantly that results fleet downtime. With the fact of increasing budget cuts and insufficient funds, force fleet managers to over utilize existing fleet that is even making the scenario even worse (NCHRP 13-04 report 2018). Transportation agencies have for many years used performance measures to help forecast and track the impacts of fleet program investments, maintenance, and operations improvements; monitor the condition of fleet assets; and gauge the management and service delivery of the agency (NCHRP Report 551 2006). The AASHTO Subcommittee on Maintenance(SCOM) has identified replacement analysis as one of the four key fleet system performance measures for State Department of Transportation for cost effective and safe fleet asset management operations. The replacement decision is primarily governed based upon a desire to reduce the fleet costs (Fan and Machemehl 2010). Therefore, it is important to have an optimum replacement cycle that will provide a deterministic schedule of planned fleet asset replacement (Fan et al. 2012). However, there is no such realistic fleet replacement guidelines in the industry to determine optimum replacement cycle (Weissmann et al. 2003). Optimum fleet replacement schedule is warranted for the following reasons but are not limited to: i) identify fleet assets targeted for replacement, ii) prioritize replacement fleets, iii) planned acquisition of new fleet asset and iv) planned way of fleet asset disposal.

This paper describes industry best practices in establishing criteria and threshold for optimum fleet replacement and presents a replacement analysis tool to develop replacement guidelines. This paper also describes an example implementation of fleet replacement analysis for a US Department of Transportation using AgileAssets Inc. Fleet Management software module. This powerful analysis tool will help the decision makers identify the short and long terms need of replacement budgets, maximize residual value of existing fleet asset and avoid unexpected fleet retention.
REPLACEMENT CRITERIA

There are 4 replacement criteria considered for the analysis. They are Useful Life, Odometer Reading, Life to Date Repair Cost, Life to Date Down Time.

*Useful life* is considered as the first criteria for the replacement analysis. Useful life is the time during which a fleet asset is expected to be usable for its primary purpose of acquisition. In other words, useful life is an estimate of the average number of years a fleet asset is considered usable before value is fully depreciated. This life span may or may not correspond with the fleet asset’s actual physical life. Useful life is an important parameter in the calculation of assets depreciation cost as well.

*Mileage* is considered as the second criteria for the replacement analysis. The mileage of the fleet asset can be determined from the Odometer reading. The Accumulated Meter reading of a fleet asset is the summation of current meter and the difference of old meter and new meter reading if any case meter swap had happened. Therefore, the accumulated meter reading is considered as one of the replacement criteria for the analysis.

*Life to Date (LTD) Repair Cost* is considered as the third criteria for the replacement analysis. LTD repair cost is the cumulative costs of all the repair activities that happened during the life of the fleet asset. These costs include labor cost, equipment cost, parts cost, commercial vendors cost, freight cost, miscellaneous cost etc. Repair activities on a fleet can occur in an agency owned on-premise repair facilities or commercial vendor owned repair facilities. The LTD repair costs include the summation of all costs (in-house and commercial repair facilities) during the life of the fleet asset regardless of type of repair facilities or service.

*Life to Date (LTD) Down Time* is considered as the fourth criteria for the replacement analysis. Downtime is the time period when a fleet asset is unavailable primarily due to maintenance (scheduled or unscheduled). LTD down time is the summation of down time (hrs or days) of a fleet asset when it is non-operational. Excessive downtime will lead to subsequent disposal of fleet assets.

REPLACEMENT THRESHOLD

The threshold for useful life is defined at the fleet asset class code level. For example, the useful life of a 1.5 TON DUMP TRUCK Class is 8 yrs whereas the useful life of a 2WD MID SIZE SUV is 12 yrs. The manufacturer recommends the expected useful life of a fleet asset however, the fleet agency can set their own threshold for defining the useful life.

The threshold for replacement mileage can be defined at the fleet class code or individual fleet asset level. If the threshold is set at the class code level, then all the fleet assets belong to that class flow the same threshold for replacement. If the threshold is set at individual asset level, then threshold set at the asset level supersedes the threshold set at the class code level.
LTD repair cost is the cumulative costs of all the repair activities that occurred during the life time of the fleet asset. Defining the benefit-cost ratio depends upon agency set ratio. For example, if the benefit cost ratio exceeds more than 50%, then that fleet asset may require immediate replacement.

DATA SET
The data set used for the analysis is from one of the US Department of Transportation’s Fleet Management system. There are a total number of 7,925 fleet assets used for the analysis which comprised of more than 600 different fleet classes. These class codes are categorized into four major classes named as Major Equipment, Minor Equipment, Fleet Cars and Vans and Minor 750 Equipment. Table 1 represents the total count of classes per major fleet classes. Only Active-in-service, on-hold, out-of-service and newly purchased assets are considered for the analysis. The assets attributes available in the replacement analysis window are fleet class codes, status, name of the fleet asset, current meter reading, accumulated meter reading, useful life, date of purchase, original Price, LTD repair Cost, LTD usage, LTD downtime, replacement due indicator etc.

Table 1 – Categorical count of fleet class

<table>
<thead>
<tr>
<th>Fleet Class Code</th>
<th>Total Count</th>
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<tbody>
<tr>
<td>Major Equipment</td>
<td>177</td>
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<tr>
<td>Minor Equipment</td>
<td>112</td>
</tr>
<tr>
<td>Fleet Cars and Vans</td>
<td>159</td>
</tr>
<tr>
<td>Minor 750 Equipment</td>
<td>147</td>
</tr>
</tbody>
</table>

EMPIRICAL ANALYSIS & RESULTS

Replacement Criteria 1: Useful Life
The purchase date of fleet asset is used to determine the current age of the fleet asset. The useful life of the asset is defined at the class code level. That means any asset that belongs to a specific class code follows the class code level threshold for useful life. The formula for calculating remaining life or exceeded life is as follows:

Remaining Life = Useful Life - Vehicle Age
Age Exceeds = Vehicle Age - Useful Life

Table 2- shows the current inventory counts per class code where asset age already exceeds more than 10 years ago compare to its respective useful life.

Table 2: Current inventory on hand per class where age exceed>10 yrs

<table>
<thead>
<tr>
<th>Fleet Class Code</th>
<th>Current Inventory</th>
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<tbody>
<tr>
<td>MOTOR GRADERS</td>
<td>23</td>
</tr>
<tr>
<td>HOPPER BOTTOM SPREADER</td>
<td>68</td>
</tr>
<tr>
<td>COMPRESSOR AIR STATIONARY ELECTRIC</td>
<td>49</td>
</tr>
<tr>
<td>GENERATOR-1-10KWPORT</td>
<td>35</td>
</tr>
<tr>
<td>FORK LIFT TRUCKS</td>
<td>28</td>
</tr>
<tr>
<td>TRAILER 15’ OR LESS</td>
<td>21</td>
</tr>
<tr>
<td>MOWER TRACTOR REAR ROTARY</td>
<td>16</td>
</tr>
</tbody>
</table>
SNOW PLOW 31
SPREADER TAILGATE 262
TRAILER DRIVE SMART 16

*Figure 1* represents a snapshot of the Fleet Replacement Analysis window. It shows a sample list of fleet assets together with class code, status, useful life, date of purchase and current age attributes. Remaining life column shows the number of years of life remaining on the fleet asset compared to the recommended useful life. The age exceeds column shows the number of years of life that has been exceeded above the recommended useful life. This information provides a clear overview of the fleet asset current replacement condition or schedule based on useful life criteria. From the snapshot, it is evident that the Backhoe (Fleet ID# 3101872) has 1-year life remaining whereas the Sweeper (Fleet ID#2900314) has already past its useful life 9 years ago, needing an immediate replacement.

![Figure 1: Replacement schedule for fleet asset based on useful life](image)

**Replacement Criteria 2: Mileage**

The accumulated meter reading represents the true mileage of the fleet asset. This is the summation of current meter and the difference of meter reading old meter and new meter reading if any meter swap had happened in the life time of the asset. Most of the cases, the accumulated meter reading is equal to the current meter reading if there has been no meter swap happened in the life of the asset. The replacement miles threshold is defined in the fleet class code level. The formula for calculating remaining miles or exceeded miles is as follows:

- Remaining Miles = Replacement Miles - Accumulated Odometer Reading
- Miles Exceeds = Accumulated Odometer Reading - Replacement Miles
Table 3 represents the current inventory counts per class code where the asset accumulated odometer reading exceeded 10,000 compare to their respective replacement miles threshold.

**Table 3: Current inventory on hand per class where mileage exceed >10000**

<table>
<thead>
<tr>
<th>Fleet Class Code</th>
<th>Current Inventory</th>
</tr>
</thead>
<tbody>
<tr>
<td>TRUCK 1 TON &amp; ABOVE FLAT W/CRANE WITH GPS</td>
<td>8</td>
</tr>
<tr>
<td>TRUCK 1 TON CREW CABS WITH GPS</td>
<td>68</td>
</tr>
<tr>
<td>TRUCK SEMI-TRACTOR</td>
<td>7</td>
</tr>
<tr>
<td>TRUCK 10 - 12 YD TANDEM DUMP</td>
<td>10</td>
</tr>
<tr>
<td>TRUCK 4 YD &amp; ABOVE DUMP SINGLE AXLE DIESEL</td>
<td>48</td>
</tr>
<tr>
<td>TRUCK AERIAL BOOM UP TO 45FT WITH GPS</td>
<td>8</td>
</tr>
<tr>
<td>TRUCK 1 TON &amp; ABOVE UTILITY WITH GPS</td>
<td>32</td>
</tr>
<tr>
<td>TRUCK 1 TON SAFETY VEHICLE WITH GPS</td>
<td>12</td>
</tr>
</tbody>
</table>

Figure 2 represents a snapshot of the Fleet Replacement Analysis window. It shows a sample list of fleet assets together with class code, status, replacement miles threshold, current odometer reading, accumulated odometer reading attributes. Remaining miles column shows the number of miles remains on the fleet asset compare to the replacement miles threshold. The Miles Exceeds column shows the miles that has been exceeded above the replacement miles threshold. This information provides a clear overview of the fleet asset current replacement condition or schedule based on replacement miles criteria. From the snapshot, it is evident that the Cumberland Tunnel class (Fleet ID# 0009826) has 57,857 miles remaining before reaching its replacement miles threshold of 125,000 whereas the Truck 1 Ton Crew Cab class (Fleet ID#0000399) has already exceeded its replacement miles threshold of 100,000 and its current accumulated mileage is 123,426 meaning 23,426 above its threshold, needing an immediate replacement.

![Figure 2: Replacement schedule for fleet asset based on replacement miles](image-url)
Replacement Criteria 3: Life to Date Repair Cost

The repair of the fleet asset can be warranted due to planned or unplanned maintenance on the fleet. The repair work can be performed in an agency owned repair facility or in a commercially owned facility. The repair cost must be tracked and summed up to calculate the total repair cost of the fleet asset regardless of the facilities (agency owned or commercial). The distinct categories of repair costs like labor cost, parts cost, freight cost, miscellaneous cost must be summed up to calculate to total repair cost of the fleet asset. Finally, all repair costs for the life of the fleet asset has been accumulated to obtain the Life to Date (LTD) repair cost.

The fleet asset inventory contains the original cost of the asset. Most of the time the original cost is equivalent to the purchase cost of the fleet asset. The information is stored in the inventory at the time of purchasing the asset. Sometimes, additional accessories have been added to the asset after the purchase that increases the value of the asset. The cost of adding accessories is tracked as capitalization cost. Therefore, the total cost of the fleet asset is the summation of original cost and capitalization cost. The LTD repair cost is divided by the total cost of the asset to calculate the Life to Date (LTD) Maintenance ratio. The formula is shown below:

$LTD\ Maint\ Ratio = LTD\ Repair\ Cost/Total\ Cost$

The value obtained from the LTD Maint Ratio column is represented in the Rep. Due Indicator column with distinct color codes based on the LTD Maint ratio range. This range of LTD Maint ratio and its corresponding color code can be altered based on agency own standard and nomenclature. Below are the color code legends for distinct range of LTD Maint ratio.

Table 4: Rep. Due Indicator Color Coding Legends

<table>
<thead>
<tr>
<th>LTD Maint Ratio</th>
<th>Color Code</th>
<th>Legend</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;0.20</td>
<td>BLUE</td>
<td></td>
</tr>
<tr>
<td>&gt;=0.20 and &lt;0.35</td>
<td>GREEN</td>
<td></td>
</tr>
<tr>
<td>&gt;=0.35 and &lt;0.50</td>
<td>YELLOW</td>
<td></td>
</tr>
<tr>
<td>&gt;=0.50</td>
<td>RED</td>
<td></td>
</tr>
</tbody>
</table>

Figure 3 represents a snap shot of the Fleet Replacement Analysis window. It shows a sample list of fleet assets together with class code, status, total cost, repair cost, LTD maint ratio attributes. LTD Maint ratio column is the ratio of life to date repair cost to fleet asset’s total cost. The Rep due indicator column represents the color coding based on the LTD maint ratio column value. This information provides a clear overview of the fleet asset current replacement condition or schedule based on LTD repair cost criteria. From the snap shot, it is evident that the Snow Plow (Fleet ID# 5506257) current LTD maint ratio is 0.33 (less than 0.35 - green colored) meaning this fleet asset doesn’t need to be replaced at this point, whereas the Spreader Tailgate class
(Fleet ID# 5505426) current LTD maint ratio is 0.44 (greater than 0.35 but less than 0.50 – yellow colored) meaning this fleet asset may require to replace in the near future when the ratio reaches to 0.50. The Trailer 15’ or less class (Fleet ID# 5400812) current LTD maint ratio is 0.59 (greater than 0.50 – red colored) meaning this fleet asset required immediate replacement.

Figure 3 : Replacement schedule for fleet asset based on life to date repair cost

The Life to date down time is calculated as the summation of all time (in hours) the fleet asset was down due to repair. The downtime of the fleet asset needs to be seen together with the LTD Maint ratio to make a better decision when asset needs to be replaced due to excessive repair costs. Setting up a threshold for LTD downtime is challenging for an agency to decide the optimum downtime of a fleet. There is no specific guidelines or threshold for downtime, so the LTD downtime is not compared to any downtime threshold.

Finally, all four replacement criteria have been incorporated simultaneously in the analysis to make a comprehensive decision to find out the candidate fleet assets that are ready to be replaced. The replacement due column displays the replacement criteria(s) based on which the fleet asset is due for replacement. For example, if a fleet asset is due for replacement based on only one replacement criteria, then the replacement due column displays value like 1 (AGE) or 1 (MILES or 1 (LTD Maint). If the fleet asset is due for replacement based on two replacement criteria at a time, then the column displays values like 2 (AGE-MILES) or 2(AGE-LTD Maint) or 2(MILES-LTD Maint). On the other hand, if a fleet asset is due for
rejection based on all replacement criteria, then the replacement due column shows values like 3 (AGE-MILES-LTD Maint). Figure 4 represents a snapshot of the replacement analysis window for few assets where the fleet is due for replacement based on one or two or all replacement criteria.

<table>
<thead>
<tr>
<th>* Equipment</th>
<th>* Equipment Class</th>
<th>Equipment Status</th>
<th>Replacement Due</th>
</tr>
</thead>
<tbody>
<tr>
<td>000034</td>
<td>TRUCK 1 TON &amp; ABOVE UTILITY WITH GPS</td>
<td>HOLD</td>
<td>2 (AGE-MILES)</td>
</tr>
<tr>
<td>0000154</td>
<td>TRUCK 1 TON &amp; ABOVE UTILITY WITH GPS</td>
<td>HOLD</td>
<td>2 (AGE-MILES)</td>
</tr>
<tr>
<td>0000172</td>
<td>TRUCK 1 TON &amp; ABOVE UTILITY WITH GPS</td>
<td>HOLD</td>
<td>2 (AGE-MILES)</td>
</tr>
<tr>
<td>0000155</td>
<td>TRUCK 1 TON &amp; ABOVE UTILITY WITH GPS</td>
<td>AVAL</td>
<td>2 (AGE-MILES)</td>
</tr>
<tr>
<td>0009757</td>
<td>TRUCK 1 TON &amp; ABOVE UTILITY WITH GPS</td>
<td>AVAL</td>
<td>2 (AGE-MILES)</td>
</tr>
<tr>
<td>009368</td>
<td>TRUCK 1 TON &amp; ABOVE UTILITY WITH GPS</td>
<td>AVAL</td>
<td>2 (AGE-MILES)</td>
</tr>
<tr>
<td>009773</td>
<td>TRUCK 1 TON &amp; ABOVE UTILITY WITH GPS</td>
<td>AVAL</td>
<td>2 (AGE-MILES)</td>
</tr>
<tr>
<td>009579</td>
<td>TRUCK 1 TON &amp; ABOVE UTILITY WITH GPS</td>
<td>HOLD</td>
<td>2 (AGE-MILES)</td>
</tr>
<tr>
<td>009633</td>
<td>TRUCK 1 TON &amp; ABOVE UTILITY WITH GPS</td>
<td>AVAL</td>
<td>2 (AGE-MILES)</td>
</tr>
<tr>
<td>00962</td>
<td>TRUCK 1 TON &amp; ABOVE UTILITY WITH GPS</td>
<td>AVAL</td>
<td>2 (AGE-MILES)</td>
</tr>
<tr>
<td>00969</td>
<td>TRUCK 1 TON &amp; ABOVE UTILITY WITH GPS</td>
<td>AVAL</td>
<td>2 (AGE-MILES)</td>
</tr>
<tr>
<td>000062</td>
<td>TRUCK 1 TON &amp; ABOVE UTILITY WITH GPS</td>
<td>AVAL</td>
<td>2 (AGE-MILES)</td>
</tr>
<tr>
<td>000072</td>
<td>TRUCK 1 TON &amp; ABOVE UTILITY WITH GPS</td>
<td>AVAL</td>
<td>2 (AGE-MILES)</td>
</tr>
<tr>
<td>000315</td>
<td>TRUCK 1 TON &amp; ABOVE UTILITY WITH GPS</td>
<td>AVAL</td>
<td>2 (AGE-MILES)</td>
</tr>
<tr>
<td>000317</td>
<td>TRUCK 1 TON &amp; ABOVE UTILITY WITH GPS</td>
<td>AVAL</td>
<td>2 (AGE-MILES)</td>
</tr>
<tr>
<td>000167</td>
<td>TRUCK 1 TON &amp; ABOVE UTILITY WITH GPS</td>
<td>AVAL</td>
<td>3 (AGE-MILES-LTD Maint)</td>
</tr>
<tr>
<td>000358</td>
<td>TRUCK 1 TON CREW CABS WITH GPS</td>
<td>AVAL</td>
<td>1 (AGE)</td>
</tr>
<tr>
<td>000179</td>
<td>TRUCK 1 TON CREW CABS WITH GPS</td>
<td>HOLD</td>
<td>1 (AGE)</td>
</tr>
<tr>
<td>000252</td>
<td>TRUCK 1 TON CREW CABS WITH GPS</td>
<td>AVAL</td>
<td>1 (AGE)</td>
</tr>
<tr>
<td>000250</td>
<td>TRUCK 1 TON CREW CABS WITH GPS</td>
<td>HOLD</td>
<td>1 (AGE)</td>
</tr>
</tbody>
</table>

Figure 4: Replacement due indicator of fleet asset from the analysis

CONCLUSIONS

Fleet performance measure has a direct relationship with replacement schedule. If fleet asset is not replaced in an optimum and timely manner; it increases operating cost, maintenance cost, replacement cost and downtime. Replacing fleet at the right time not only reduces the overall cost of ownership but also help determining the correct size of the fleet. This paper tries to elaborate industry best practices in establishing fleet asset replacement criteria and its threshold and presents a replacement analysis tool to develop replacement guidelines for the agency. The 4 criteria considered for the analysis: Useful Life, Odometer Reading, Life to Date Repair Cost, Life to Date Down Time are the most important criteria to determine the replacement life cycle of a fleet asset. Each of the replacement criteria can be individually and combinedly sighted before making the replacement decision. The State Department of Transportation (DOT), Public & Private Agency, Cities and Counties can perform their own fleet replacement analysis by following the above replacement guidelines. This powerful analysis tool will help the decision makers identify the short and long terms need of replacement budgets, maximize residual value of existing fleet asset and avoid unexpected fleet retention.
ACKNOWLEDGEMENTS

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REFERENCES


4. NCHRP Research Report 551 Performance Measures and Targets for Transportation Asset Management


# PAPER TITLE
Repeatability and Agreement of Pavement Texture Surface Profiles using Static and Walking-Speed High-Density Laser-Triangulation Devices

## TRACK
Pavements

## AUTHOR (Capitalized Family Name)

<table>
<thead>
<tr>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
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<tbody>
<tr>
<td>Vincent Bongioanni, P.E.</td>
<td>PhD Candidate</td>
<td>Virginia Tech</td>
</tr>
</tbody>
</table>

## CO-AUTHOR(S) (Capitalized Family Name)

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<thead>
<tr>
<th>POSITION</th>
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<th>COUNTRY</th>
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</thead>
<tbody>
<tr>
<td>Kyle Maeger</td>
<td>Graduate Student</td>
<td>Virginia Tech</td>
</tr>
</tbody>
</table>

## E-MAIL (for correspondence)
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## KEYWORDS:

- Macrotexture
- Pavement Profile
- MPD
- RMS
- Surface

## ABSTRACT:

A road’s surface affects vehicle/pavement interactions such as wet and dry friction, splash and spray, road and vehicle noise, and rolling resistance. The ability to accurately measure surface profiles is paramount in understanding and quantifying these interactions. This study evaluated three walking-speed and three static non-contacting laser triangulation texture scanners in their ability to produce repeat measurements and the agreement between devices on the same pavement surface. Several readings were made of the same pavement sample and repeatability coefficients were calculated. A limits of agreement analysis was performed on devices to evaluate the interchangeability of the devices. The results demonstrate good repeatability for the two static devices tested and for the walking-speed devices tested. In terms of device agreement, it was found that the results from the static devices tested should not be used interchangeably in most cases. For the walking-speed devices, some combinations of device and pavement texture group indicated results can be used interchangeably while, in other combinations, they should not.
Device Agreement and Repeatability of Static and Walking-Speed Pavement Texture Measurement Devices

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1 INTRODUCTION

Pavement surface texture is a critical factor in several vehicle/road responses. PIARC (The World Road Association) defines macrotexture as “surface irregularities of a road pavement with horizontal dimensions ranging between 0.5 mm and 50 mm and vertical dimensions between 0.2 and 10 mm” (PIARC 2016). Smaller wavelengths are a pavement’s "microtexture" and longer wavelengths are "megatexture" then road "roughness." Important safety concerns such as friction and splash and spray have been found to be related to the pavement’s surface profile (Wambold et al. 1995; Weir et al. 1978). The effect of texture is compounded when water (even a thin film) is present. Several studies (Parry and Viner 2005; Roe et al. 1998) have shown a correlation between texture and vehicle crashes in wet weather. Larger voids beneath the tire tend to evacuate water allowing a tire’s contact patch to come in direct contact with the pavement to avoid hydroplaning. Increased tire contact allows for greater tire/pavement adhesion (a function of the pavement’s microtexture) as well as friction through hysteresis (energy loss due to asymmetrical deformation of the tire). Hysteresis has an exponential increase with vehicle speed, accounting for 95 percent of available friction at speeds above 65 mph (Hall et al. 2009). Pavement texture has also been shown to be a contributing predictor of rolling resistance (Sandberg et al. 2011) and road noise (Descornet et al. 2000) which carry far-reaching economic and social impacts.

In order to characterize a pavement’s surface, non-contacting laser triangulation devices are often used to record the surface profile of the pavement. These devices are typically favored over physical media tests (i.e., sandpatch) that are cumbersome and prone to operator error. Two-dimensional (2-D) profilers are common in gathering pavement texture data, given the most prevalent technologies used to gather texture data are 2-D. However, three-dimensional (3-D) surface maps can also be produced using similar technology. A distinct advantage these devices have over 2-D profiles is a larger quantity of data gathered over a given area. 3-D datasets can allow further inferences on a pavement’s true character by providing information on the orientation, spacing, and directionality of surface asperities.

Currently, the ROSANNE project is assessing the practicality of implementation of 3-D texture measurements on European roads (Goubert 2016). The decision to adopt 3-D measurements will be based on the further investigation on the usefulness of the data in its application to pavement/tire interactions. Additionally, the International Organization for Standardization (ISO) is in the process of a major revision to International Standard ISO 13473-1 (1997) for the measurement of macrotexture. ISO identified the need for such an update due to wide variability and poor repeatability between various measurement devices used in the field today.

2 OBJECTIVE

The objective of this work is to assess the repeatability (ability to produce equal measurements) and agreement (analysis of difference in measurements between devices) of several non-contacting laser triangulation pavement surface profilers. These devices are both placed (static) and pushed (walking-speed) along the pavement surface. These devices could be considered reference devices for network-level macrotexture measurement devices.

3 METHODOLOGY

The Experiment was carried out at the Heavy Vehicle Simulator (HVS) and the test track at the Virginia Tech Transportation Institute (VTTI). These locations were used because the enclosed environment of the HVS shielded the measurement device from unwanted weather effects and measurements could be made on the test track without the influence or danger of traffic. The test track is a 2.2-mile loop dedicated to research and is
located in Blacksburg, Virginia. It offers a variety of surfaces, including Dense Graded Hot Mix Asphalt (DGHMA), an Open-Graded Friction Course (OGFC), proprietary High-Friction Surface Treatments (HFST), as well as grooved, tined, or ground Portland Cement Concrete (PCC) sections. The HVS was not running and no vehicles were moving on the test track at the time of the experiment to ensure the testing apparatuses were unaffected by potential movement. Figure 1 shows the experiment setup within the HVS. The test track was marked with chalk line for walking-speed devices to follow and painted with boxed areas for static device placement (Figure 2). Walking-speed devices collected data along the chalk line continuously and static devices collected a single profile in each of 5 boxes painted on each pavement section.

Properties for the static devices can be found in Table 1. The repeatability of devices 6 and 12 was tested inside the HVS. The repeatability of device 8 was not determined due to unavailability at the time of testing. For device 12, scans were made at maximum resolution, with a 0.00635 mm longitudinal (direction of traffic) sampling interval and a 0.0247 mm transverse sampling interval. This resulted in a 3-D pavement surface point cloud with 16,380 x 2,917 (over 47 million) texture height measurements. Five replicates were created by pressing the “scan” button on the top of the machine five times while the device remained in place. Figure 3 shows an example of the surface profile measured.
Table 1 - Static measurement device characteristics

<table>
<thead>
<tr>
<th>Device ID</th>
<th>Laser type</th>
<th>Measurement Patch</th>
<th>Sample Distance</th>
<th>Vertical Resolution</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>Single spot</td>
<td>300mm circular</td>
<td>0.087mm</td>
<td>0.003mm</td>
</tr>
<tr>
<td>8</td>
<td>Line laser</td>
<td>101.6 x 101.6mm</td>
<td>0.0496mm L x</td>
<td>0.01mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.0415mm W</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>Single spot</td>
<td>107.95mm L</td>
<td>0.00635mm L x</td>
<td>0.003mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>72.01mm W</td>
<td>0.0247mm W</td>
<td></td>
</tr>
</tbody>
</table>

Figure 3 - Example pavement surface profile from Device 12

Pavement profiles are analyzed for their effect on vehicle/road interactions by representing key characteristics in pavement surface as parameters. The software package for device 12 is capable of calculating a wide array of these parameters from a given measured profile. For example, in the United States, a pavement’s Mean Profile Depth (MPD) is typically reported for a given pavement. In this study, repeatability values for device 12 are given for the following parameters to facilitate interpretation of results by agencies that may use them.

Table 2 - Summary of parameters used for device 12

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Reference/Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>MPD – Mean Profile Depth</td>
<td>ASTM E1845 (2015); ISO 13473-1 (1997)</td>
</tr>
<tr>
<td>ETD - Estimated Texture Depth</td>
<td>ASTM E1845 (2015); ISO 13473-1 (1997)</td>
</tr>
<tr>
<td>RMS - Root Mean Square</td>
<td>ISO 13473-2</td>
</tr>
<tr>
<td>Length</td>
<td>Length of each scan line (accounting for changes in elevation)</td>
</tr>
<tr>
<td>Length Ratio</td>
<td>Ratio of profile Length to device scan length (107.95mm)</td>
</tr>
<tr>
<td>$R_q$ – Mean Square Roughness</td>
<td></td>
</tr>
<tr>
<td>$R_{sk}$ – Skewness</td>
<td></td>
</tr>
<tr>
<td>$R_{ku}$ – Kurtosis</td>
<td></td>
</tr>
</tbody>
</table>

The mean profile depth (MPD) was selected as the parameter to represent the macrotexture measured by all walking-speed devices as it is widely used in both the US and abroad. The characteristics for the walking-speed devices used can be seen in Table 3 below.

Table 3 - Walking speed device characteristics

<table>
<thead>
<tr>
<th>Device ID</th>
<th>Laser Type</th>
<th>Sample distance</th>
<th>Vertical Resolution</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>Line Laser</td>
<td>1 mm (transverse)</td>
<td>&lt; 0.05mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1 mm (longitudinal)</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Single Spot</td>
<td>1 mm</td>
<td>0.005mm</td>
</tr>
<tr>
<td>11</td>
<td>Line Laser</td>
<td>0.3 mm (transverse)</td>
<td>0.015 - 0.040mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.5 mm (longitudinal)</td>
<td></td>
</tr>
</tbody>
</table>
For the repeatability analysis of the walking-speed devices and the limits of agreement analysis of all static and walking-speed devices, the test track was used. Representative surfaces of the test track were selected to provide a variety of texture types and orientations for the analysis. The pavement sections tested are listed in Table 4.

<table>
<thead>
<tr>
<th>Section</th>
<th>Material</th>
<th>Surface</th>
<th>Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SRB</td>
<td>PCC</td>
<td>Transverse Grooved CRCP</td>
<td>610</td>
</tr>
<tr>
<td>PCC 2</td>
<td>PCC</td>
<td>Long diamond ground JPCP</td>
<td>178</td>
</tr>
<tr>
<td>PCC 1f</td>
<td>PCC</td>
<td>Long grooved &amp; ground</td>
<td>162</td>
</tr>
<tr>
<td>PCC 1d</td>
<td>HFST</td>
<td>Cargill Safe Lane</td>
<td>30</td>
</tr>
<tr>
<td>PCC 1b</td>
<td>HFST</td>
<td>EP-5</td>
<td>30</td>
</tr>
<tr>
<td>PCC 1a</td>
<td>PCC</td>
<td>Transverse tined CRCP</td>
<td>69</td>
</tr>
<tr>
<td>L1</td>
<td>AC</td>
<td>SMA-12.5</td>
<td>91</td>
</tr>
<tr>
<td>K</td>
<td>AC</td>
<td>Open Graded Friction Course</td>
<td>92</td>
</tr>
<tr>
<td>J</td>
<td>AC</td>
<td>SM-9.5D</td>
<td>89</td>
</tr>
<tr>
<td>H</td>
<td>AC</td>
<td>SM-9.5D</td>
<td>92</td>
</tr>
</tbody>
</table>

Data Preparation

MPD was selected as the parameter to represent the various pavement surface types tested for most of the repeatability and agreement analysis in this work. As noted in various standards for calculating MPD (17; 18), outliers in the data must first be removed. This is especially important for MPD calculations as they are sensitive to large data variations away from the mean. Data from static devices was treated using the methods implemented by the manufacturer’s software. All data from walking-speed devices were treated with the adaptive outlier removal routine as described by Katicha et al. (19). The authors wanted to use a consistent approach to outlier removal for all walking-speed datasets. The false discovery rate method employed is a statistics-based approach that adapts to the given dataset to select an appropriate threshold that guarantees that no more than a certain percentage (10% in this case) of outliers are incorrectly identified. Due to its statistical nature, large datasets should be used, so consecutive rows of line laser data were concatenated according to equation 1 to create a dataset similar in size to those of the single-spot lasers for their outlier elimination.

\[
\begin{bmatrix}
  h_{1,1} & h_{1,2} & \ldots & h_{1,n} \\
  \vdots & \ddots & \vdots & \vdots \\
  h_{m,1} & h_{m,2} & \ldots & h_{m,n}
\end{bmatrix}
\rightarrow
\begin{bmatrix}
  h_{1,1} & \ldots & h_{m,n}
\end{bmatrix}
\]

To minimize edge effects in the concatenation, a simple linear regression subtracted from the row of interest suppressed the slope and set the mean of the row to zero. After outlier removal, the dataset is reconstructed to the original matrix dimensions. Figure 4 shows a sample 3-D profile before and after outlier removal using this method.

![Figure 4 - Example raw (a) line laser profile and (b) profile with outliers removed](image)
After outlier removal, all walking-speed data were filtered to ensure only the wavelengths of interest for macrotexture are evaluated. A low-pass, lowest-order Butterworth Infinite Impulse Response filter was designed for each device based on the sampling interval used according to the guidance in ASTM E1845 (17). All long wavelength trends (i.e., grade of the road, slope due to position of the device relative to the surface) of the 100mm MPD base lengths were removed. This was accomplished by subtracting a first-order regression line from the profile of each base length, resulting in a zero-mean, zero-slope (detrended) base length. Finally, MPDs were calculated for every 100 mm of longitudinal travel in the left wheelpath according to ASTM E1845. For the walking-speed line lasers, MPD values were calculated in the transverse direction and then arithmetic means were calculated to harmonize the reporting length of the line laser with the results of the single spot lasers. For example, MPDs were calculated for each transverse line laser scan and all MPD values within the 100mm of interest. This means the data from more than 100 profiles from the line laser walking-speed devices are averaged for every MPD reported by the walking-speed device with the single spot laser.

Device Comparison

To overcome the limitations of other forms of device comparison (i.e., ANOVA, correlation, and harmonization), the repeatability of each device and the agreement in measurement between device pairs (which also sheds light on an instrument’s bias) were evaluated. Conducting an ANOVA can show if means are equal for multiple runs of a device, where a high \( p \)-value corresponds to a failure to reject the null hypothesis, indicating that the means of repeated measurements taken are equal. However, this approach is limited when comparing devices one with another because a device with high variance in collected data tolerates a more substantial difference in means before rejecting the null hypothesis of equal means than a device with lower variance.

Correlation analysis for the purpose of testing a device’s repeatability can also be misleading as it shows only the strength of the relationship between devices, but does not quantify the differences (or, conversely, agreement) between devices. Harmonization studies have been used in the past to compare a device against a “ground truth” in order to force measurements to be more similar to another device. This is difficult for a pavement’s macrotexture as the true value of macrotexture is unknown for the surfaces measured; in other words, there is no “ground truth.” Furthermore, these studies have been difficult to replicate under differing experimental conditions (7-9). The methods employed to quantify repeatability and agreement are described below.

Repeatability

Repeatability determines the extent to which a device can reproduce its previous results. In this study, this is accomplished by quantitatively by calculating a device’s repeatability coefficient. A repeatability coefficient is superior to a coefficient of correlation as the former provides a specific quantity (i.e., difference in MPD measured in millimeters), whereas the latter only provides a proportion of the relationship (1.0 being a perfect relationship) between the repeat runs of the device. With the specific quantity provided by the repeatability coefficient, engineering judgement can be used to determine if the device has sufficient repeatability for the planned purpose. For example, a device with a repeatability coefficient of 0.05mm may be deemed sufficient for measuring macrotexture as this is within the resolution required to delineate between investigatory and intervention levels which is often given in tenths of a millimeter. The repeatability coefficient is derived from the device’s mean square error (MSE) from an ANOVA for several runs over the same pavement section, where MSE is essentially the variance of the device.

Repeat measurements are made on the same subject under identical conditions. In this study, this was achieved by measuring the same patch of pavement with static devices in the HVS and the same sections of the test track with walking-speed devices. From these measurements, the within-subject variance (MSE), or, standard deviation (SD) which is the square root of MSE is calculated. Since we are interested in the amount measurements change between replicates, the differences between these SDs are of particular importance and can be calculated per Bartlett and Frost (2008):

\[
SD \text{ of differences between measurements} = \sqrt{2} \times SD
\]  

(2)
If we expect 95% of the differences between measurements to be within two SDs of each other, equation 1 then becomes the repeatability coefficient as defined by the British Standards Institution (British Standards Institution 1979):

\[
\text{Repeatability coefficient } (c_r) = 1.96 \times \sqrt{2} \times SD
\]  

As multiple runs were made with each walking-speed device, SD (the within-device standard deviation) in equation 2 is calculated by taking the square root of the MSE after performing an ANOVA. The interpretation of the test is that two measurements made on a subject by a device should differ by no more than the repeatability coefficient 95% of the time, assuming a normal distribution of differences between measurements (20). For the walking-speed devices, the ANOVA was conducted for the devices with pavement section as the input and MPD as the model effect. For all the analyses, 1-meter aggregated data collected over the pavement sections were averaged to a single average MPD for each pavement section for each of the three runs.

**Device Agreement**

Limits of agreement (LOA) is presented by Bland and Altman (12) as a superior device comparison method for correlation coefficients because a strong correlation between devices does not necessarily guarantee strong agreement between them. This method explores the differences between measurements made by any two devices and quantifies these differences against the mean of the measurements made. Three runs of walking-speed data were collected over the 10 pavement sections as shown Table 4.

\[
\text{LOA} = 1.96 \times S_c
\]  

where:

- \( S_c \) is the corrected standard deviation of differences = \( \sqrt{S_D^2 + f_1 \cdot S_1^2 + f_2 \cdot S_2^2} \);
- \( S_D \) = standard deviation of the difference between the mean of the runs for each road section by two devices compared;
- For the walking-speed devices \( f_1 \) and \( f_2 = 1 - \frac{1}{m} \) (\( m \) is the number of runs for each section). 3 runs per section; therefore, \( f_1 = f_2 = 0.67 \).
- \( S_1 \) and \( S_2 \) are the variances of the devices (MSE determined from an ANOVA of the device with the pavement section as the model input and average MPD as the response). For comparison of static devices \( S_c \) is simplified to just \( S_D \). This is because only one measurement was taken at each measurement location along the pavement sections, therefore the within-device variance cannot be included in the calculation.

The boundaries for LOA are the mean of the differences for the devices over the sections ± the LOA. The LOA are plotted with the mean of the two devices’ measurements on the x-axis and the difference between the devices on the y-axis.

**4 RESULTS**

To begin the analysis, a two-way factorial ANOVA was performed for the walking-speed devices. In this analysis, the device, pavement section, and interaction term of device*section are all used as model effects with average MPD for the section as the response. The resulting \( p \)-values (all < 0.0001) indicated that all model effects were significant to the analysis.

The effects test of the device was used to determine if any of the devices differ. From the two-way ANOVA, a Tukey’s Honest Significant Difference (HSD) test with connecting letters report from JMP software was used to make a preliminary evaluation of device agreement. It should be noted that Tukey’s HSD is a very sensitive test and small differences may show as disagreement of an entire device, as shown in Table 5. Note that levels not connected by the same letter are significantly different in this case indicating all the walking-speed devices are significantly different. Further light is shed in subsequent pairwise LOA comparisons in this work.
Table 5 - Tukey HSD analysis of walking-speed devices

<table>
<thead>
<tr>
<th>Level</th>
<th>Least Squares Mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>11 A</td>
<td>1.400</td>
</tr>
<tr>
<td>7 B</td>
<td>1.232</td>
</tr>
<tr>
<td>9 C</td>
<td>1.170</td>
</tr>
</tbody>
</table>

Repeatability

Device 12

The average output values were calculated by the device software. The various parameters are calculated individually for the 2,917 scan lines and the arithmetic mean is made to represent the total pavement surface scanned. These average output values (one for each of the five replicates) of each parameter are used in the calculation of the repeatability coefficients given in Table 6. These coefficients represent the maximum difference that can be expected from the device on the same pavement sample 95 percent of the time. For example, if an average MPD of 0.80 mm is measured by the device found for the pavement, 95 percent of measurements will be within the range 0.798 to 0.802 mm based on these results. Average values represent how a road agency may gather and summarize data for a network of roads in their jurisdiction.

Table 6 – Repeatability coefficient of mean results output by device 12

<table>
<thead>
<tr>
<th>Parameter</th>
<th>cr</th>
</tr>
</thead>
<tbody>
<tr>
<td>MPD</td>
<td>0.0021</td>
</tr>
<tr>
<td>ETD</td>
<td>0.0017</td>
</tr>
<tr>
<td>RMS</td>
<td>0.0023</td>
</tr>
<tr>
<td>$R_d$</td>
<td>0.0021</td>
</tr>
<tr>
<td>$R_d$</td>
<td>0.0027</td>
</tr>
<tr>
<td>$R_{sk}$</td>
<td>0.0292</td>
</tr>
<tr>
<td>$R_{ku}$</td>
<td>0.2042</td>
</tr>
<tr>
<td>Length</td>
<td>1.8909</td>
</tr>
<tr>
<td>Length Ratio</td>
<td>0.0182</td>
</tr>
</tbody>
</table>

Greater granularity is possible from the data collected by device 12. An analysis of the full range of data available for all parameters of the five replicates was made. An example distribution of results is shown in Figure 5. Statistics for the remaining parameters is summarized in Table 7 below.

Figure 5 - Histogram and boxplot of all RMS values gathered for the subject pavement sample
Table 7 - Summary Statistics of Parameters Collected for five Replicates

<table>
<thead>
<tr>
<th></th>
<th>MPD</th>
<th>ETD</th>
<th>RMS</th>
<th>R_a</th>
<th>R_q</th>
<th>R_sk</th>
<th>R_ku</th>
<th>Length</th>
<th>Length Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>0.7969</td>
<td>0.8375</td>
<td>0.8317</td>
<td>0.6324</td>
<td>0.8544</td>
<td>-1.9634</td>
<td>4.8957</td>
<td>178.9857</td>
<td>1.7209</td>
</tr>
<tr>
<td>Std Dev</td>
<td>0.2165</td>
<td>0.1732</td>
<td>0.1760</td>
<td>0.1531</td>
<td>0.1858</td>
<td>0.6444</td>
<td>4.3443</td>
<td>25.2706</td>
<td>0.2430</td>
</tr>
<tr>
<td>Max</td>
<td>1.5489</td>
<td>1.4391</td>
<td>1.2914</td>
<td>1.1007</td>
<td>1.3504</td>
<td>-0.3437</td>
<td>34.0445</td>
<td>330.1283</td>
<td>3.1741</td>
</tr>
<tr>
<td>Q75</td>
<td>0.9202</td>
<td>0.9362</td>
<td>0.9559</td>
<td>0.7412</td>
<td>0.9820</td>
<td>-1.5045</td>
<td>6.7330</td>
<td>192.2967</td>
<td>1.8489</td>
</tr>
<tr>
<td>Q50</td>
<td>0.8089</td>
<td>0.8471</td>
<td>0.8499</td>
<td>0.6921</td>
<td>0.8781</td>
<td>-1.8920</td>
<td>3.7577</td>
<td>174.3367</td>
<td>1.6762</td>
</tr>
<tr>
<td>Q25</td>
<td>0.6342</td>
<td>0.7073</td>
<td>0.6989</td>
<td>0.5083</td>
<td>0.7248</td>
<td>-2.3332</td>
<td>2.0146</td>
<td>160.9283</td>
<td>1.5473</td>
</tr>
<tr>
<td>Min</td>
<td>0.3903</td>
<td>0.5122</td>
<td>0.4156</td>
<td>0.2917</td>
<td>0.4366</td>
<td>-4.9160</td>
<td>-1.1910</td>
<td>134.9797</td>
<td>1.2978</td>
</tr>
</tbody>
</table>

Repeatability coefficients were then calculated for each of the five replicates across all scan lines. For example, \( c_r \) was calculated using the methodology explained above for the within-subject variability of the five MPD values calculated for scan line one. The was repeated for scan line two through 2,917. Results are summarized in Table 8. This is useful when considering using less scan lines to represent a pavement sample. In this case, the averaging effect (which decreases variability) of increased scan lines is reduced.

Table 8 - Summary Statistics of \( c_r \) for each scan line of five Replicates

<table>
<thead>
<tr>
<th></th>
<th>( c_r ) of MPD</th>
<th>( c_r ) of ETD</th>
<th>( c_r ) of RMS</th>
<th>( c_r ) of ( R_a )</th>
<th>( c_r ) of ( R_q )</th>
<th>( c_r ) of ( R_{sk} )</th>
<th>( c_r ) of ( R_{ku} )</th>
<th>Length</th>
<th>Length Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>0.0166</td>
<td>0.0133</td>
<td>0.0264</td>
<td>0.0127</td>
<td>0.0262</td>
<td>0.1516</td>
<td>1.1232</td>
<td>14.0076</td>
<td>0.1347</td>
</tr>
<tr>
<td>Std Dev</td>
<td>0.0185</td>
<td>0.0148</td>
<td>0.0325</td>
<td>0.0125</td>
<td>0.0313</td>
<td>0.2226</td>
<td>2.0068</td>
<td>12.5698</td>
<td>0.1209</td>
</tr>
<tr>
<td>Max</td>
<td>0.2520</td>
<td>0.1800</td>
<td>0.2232</td>
<td>0.1119</td>
<td>0.2166</td>
<td>1.9568</td>
<td>24.3051</td>
<td>95.4695</td>
<td>0.9179</td>
</tr>
<tr>
<td>Q75</td>
<td>0.0197</td>
<td>0.0158</td>
<td>0.0335</td>
<td>0.0157</td>
<td>0.0330</td>
<td>0.1700</td>
<td>1.1843</td>
<td>18.2823</td>
<td>0.1758</td>
</tr>
<tr>
<td>Q50</td>
<td>0.0010</td>
<td>0.0088</td>
<td>0.0143</td>
<td>0.0086</td>
<td>0.0147</td>
<td>0.0714</td>
<td>0.4224</td>
<td>9.9700</td>
<td>0.0959</td>
</tr>
<tr>
<td>Q25</td>
<td>0.0067</td>
<td>0.0054</td>
<td>0.0066</td>
<td>0.0049</td>
<td>0.0070</td>
<td>0.0326</td>
<td>0.1579</td>
<td>5.4256</td>
<td>0.0522</td>
</tr>
<tr>
<td>Min</td>
<td>0.0011</td>
<td>0.0009</td>
<td>0.0005</td>
<td>0.0004</td>
<td>0.0006</td>
<td>0.0027</td>
<td>0.0076</td>
<td>0.5998</td>
<td>0.0058</td>
</tr>
</tbody>
</table>

Device 6 (circular measurement path)

This device has a circular measurement path made with a single-spot laser. The device was tested for repeatability in three different locations within the HVS. The first location was in the middle of the traffic pattern for the test wheel of the HVS. This location is most similar to the conditions in which the device would be used in the field. Five tests were made with device 6, each time pressing the measurement button without moving the device. Results of the repeatability test for MPD and RMS (the two parameters output by the device) are found in Table 9 below. The second test location was accomplished by using the same marked location but picking up and replacing the device between measurements. This simulates the practice of repeat measurements over time. Maximum care was taken to place the device in the same location according to marks made on the pavement. However, it can be seen that the repeatability of the device decreases as these small changes in measurement location change. The third test location was in an area of the HVS that receives no traffic by the test wheel. This can be considered virgin pavement and demonstrates the small difference between virgin and trafficked pavement when compared to the difference between repeat tests and repeat tests when the device is replaced each time.

Table 9 - Summary of \( c_r \) for Device 6

<table>
<thead>
<tr>
<th>Test Location</th>
<th>( c_r ) of MPD</th>
<th>( c_r ) of RMS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traffic</td>
<td>0.05</td>
<td>0.17</td>
</tr>
<tr>
<td>Traffic – device replaced</td>
<td>0.14</td>
<td>0.20</td>
</tr>
<tr>
<td>Non-traffic</td>
<td>0.02</td>
<td>0.15</td>
</tr>
</tbody>
</table>
Walking-speed devices

The calculated coefficients of repeatability for the walking-speed devices are found in Table 10. \( C_r \) for all devices tested was in a similar range (from 0.025 to 0.054 mm), meaning that on 95% of occasions the measurements on pavement types similar to those tested by the devices used will differ by no more than the repeatability coefficient. These coefficients are for all pavement surface types tested.

<table>
<thead>
<tr>
<th>MSE</th>
<th>Device 7</th>
<th>Device 9</th>
<th>Device 11</th>
</tr>
</thead>
<tbody>
<tr>
<td>( C_r )</td>
<td>0.025</td>
<td>0.027</td>
<td>0.054</td>
</tr>
</tbody>
</table>

**Device Agreement**

Static Devices

For the static devices, the pavement tested was the same as the pavement test for the walking-speed devices. This was done to provide a variety of surfaces to test upon to better understand the agreement between devices on different types of pavement. Five locations were identified along the length of the section and marked by a 400mm x 400mm box in which the static measurement devices were placed. The results of the LOA analysis are found in Table 11 below. Note device 12 (a single-spot device) was rotated 90° on some surfaces (denoted by a “T”) with directional texturing to demonstrate the effect of said texturing on parameter calculation. For example, the agreement between devices 12 and 8 improves for the longitudinal sections when device 12 is placed with the scan lines traveling longitudinally along the pavement which captures the same pavement profile as device 8 (a static line-laser device) for each of its scan lines. Similarly, the agreement between devices 6 (a device that scans in a circular pattern) and 12 improves on the longitudinally-textured pavement when device 12 is set transversely (with the scanning line going across the grooves of the pavement texture) which is more similar to several of the arcs measured by device 6. This can most easily be seen when the means of each device are plotted against the difference between means of each device. Note the large band of agreement for devices 6 and 12 (Figure 6a) and the much more narrow band (Figure 6b) of devices 6 and 12T.

![Figure 6 - Comparison of LOA of device 6 compared to device 12 oriented in the longitudinal (a) and transverse (b) directions](image)

The analysis was initially carried out for all pavement types tested, however, it was noted that the directional texturing of many of the PCC pavements would have an effect on the macrotexture parameters calculated. Hence, the pavements were divided into four groups as seen in Table 11 below. Random texturing covers all AC pavements and PCC 1b and 1d as these are PCC pavements with a treatment of small aggregate at the surface. The longitudinal sections are comprised of PCC pavement with longitudinal grooves and/or diamond grinding (sections PCC 2 and 1f). The transverse pavements are those with transverse tines (sections SRB and PCC1a).
Table 11 - Summary of LOA analysis for static devices

<table>
<thead>
<tr>
<th>Device Pair</th>
<th>All</th>
<th>Random</th>
<th>Longitudinal</th>
<th>Transverse</th>
</tr>
</thead>
<tbody>
<tr>
<td>6-8</td>
<td>0.094</td>
<td>0.935</td>
<td>-0.111</td>
<td>0.374</td>
</tr>
<tr>
<td>12-8</td>
<td>-0.068</td>
<td>0.326</td>
<td>-0.067</td>
<td>0.333</td>
</tr>
<tr>
<td>12T-8</td>
<td>0.135</td>
<td>0.922</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>6-12</td>
<td>0.163</td>
<td>0.875</td>
<td>-0.045</td>
<td>0.217</td>
</tr>
<tr>
<td>6-12T</td>
<td>-0.040</td>
<td>0.207</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Walking-speed devices

The mean of all 1-meter MPDs for each run of a particular pavement section was calculated. The results are plotted in Figure 7 for each of the devices tested. The next task is to quantify the agreement (or lack thereof) between devices.

![Figure 7 - Mean walking-speed MPDs calculated for each pavement section tested](image)

The LOAs for each of the walking-speed devices was calculated as previously described. A summary of these results for the four pavement surface texture groups is found at Table 12. In most cases, the agreement between the two line laser sensors (devices 7 and 11) was closer than for the single-spot devices. In most cases, however, the agreement is wide enough to indicate that the measurements from the devices should not be used interchangeably one with another for the surfaces tested.
### Table 12 - Summary of LOA analysis for walking-speed devices

<table>
<thead>
<tr>
<th>Device Pair</th>
<th>Texture Type</th>
<th>All</th>
<th>Random</th>
<th>Longitudinal</th>
<th>Transverse</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean of Difference</td>
<td>LOA</td>
<td>Mean of Difference</td>
<td>LOA</td>
<td>Mean of Difference</td>
</tr>
<tr>
<td>9-11</td>
<td>-0.23</td>
<td>1.17</td>
<td>-0.12</td>
<td>0.20</td>
<td>-1.19</td>
</tr>
<tr>
<td>9-7</td>
<td>-0.06</td>
<td>1.06</td>
<td>0.02</td>
<td>0.06</td>
<td>-0.88</td>
</tr>
<tr>
<td>11-7</td>
<td>0.17</td>
<td>0.25</td>
<td>0.14</td>
<td>0.22</td>
<td>0.31</td>
</tr>
</tbody>
</table>

### 5 CONCLUSIONS

- The MPD repeatability coefficient for Device 12 on the pavement tested in the HVS was found to be 0.0021 mm.
  - This is approximately one-quarter of one percent of the mean value measured.
  - The mean repeatability coefficient was 0.0166 mm for MPD when each scan line is considered separately.
  - These values are quite good as the threshold and investigatory levels for pavement macrotexture for several agencies (Austroads AP-T290-15 2015; New Zealand Transport Agency 2013) is given with one decimal place of precision.
  - This means the variance in repeat measurements would not affect decision-makers confidence in the reading for the pavement measured in this study.
  - All other parameters calculated showed acceptable repeatability

- The MPD repeatability coefficient for device 6 (a single-spot laser device that follows a circular path) was found to be 0.05mm in the trafficked area of the HVS when repeatedly tested without moving the device. This means that 95% of all MPD measurements made with the device on this pavement will be within this tolerance.
  - When the device is picked up and replaced (to replicate field conditions) the repeatability coefficient increases to 0.14mm.
  - The field-simulated repeatability coefficient may affect decision making as the mean value of MPDs calculated may be off by greater than the resolution needed to determine if a pavement has reached an investigatory or intervention level.
  - In all three test scenarios MPD was more repeatable than RMS

- The MPD repeatability of each of the walking-speed devices tested was found to be less than 0.05mm, meaning the variance in repeat measurements would not affect decision-makers confidence in the reading for any of the pavement types measured in this study.

- The limit of agreement of all static devices tested (by MPD) on the ten pavement types tested was between 0.2 - 0.4mm for randomly-textured surfaces, 0.3 - 0.7mm for longitudinally textured pavements, and for transversely-texture sections (i.e., tines) was between 0.1 - 0.3mm. This means in most cases, the devices should not be used interchangeably for one another if, in the engineer’s judgement, the differences in measurements should not be greater than these ranges.

- The LOA for all of the walking-speed device MPDs was between 0.06mm and 0.2mm for randomly-textured sections, 0.07 to 1.2mm for longitudinally-texture sections (i.e., grooves or diamond grinding), and 0.3 to 0.8mm for transversely-tined PCC sections.
  - In general, agreement was better when comparing the two line laser sensors with one another than some combination of the line lasers and the single-spot device with the line laser
  - The interpretation of these results is that in some cases, the devices may be used interchangeably with one another (i.e., those cases where the agreement is better than within 0.1mm MPD) and in other cases they should not when the true measurement is unknown
REFERENCES


**ABSTRACT:**
Road infrastructure is a critic for Indonesia’s development, but the state budget is not enough to fully fund road infrastructure needs. The government must innovate to overcome this funding shortfall. One innovation is Availability Payment (AP), a form of Public-Private Partnership (PPP), where all costs for construction, operation and maintenance are paid up-front by the private sector partner and the government partner agency pays periodical payments as long as the public service is provided until the end of the concession period. This scheme offers a practical solution to the state budget, and it increases private sector involvement in delivering public infrastructure in Indonesia. The PPP-AP scheme is being considered for producing national non-toll road corridors for the first time in Indonesia. The proposed corridors are the Eastern Sumatera Road Corridor and the Trans-Papua Road Corridor. There are many aspects to be considered in preparing for a PPP-AP scheme, including defining the public service benefit (scope and quality of the road asset created and maintained), legal issues, stakeholders, risk allocation and mitigation, future managerial challenges, and value for money. By implementing these PPP-AP schemes, Indonesia intends to provide a high-quality public service and value for public funds by the well-governed involvement of the private sector.
I INTRODUCTION

Adequate and reliable connectivity is a crucial factor to achieve robust economic growth in any country. The road network provides the basis for connectivity that gives accessibility and mobility for people, goods and services through passenger and freight transportation. The government in every country has the responsibility to develop and implement a policy that will provide sufficient good road infrastructure to enable economic development by to developing a better network system and maintaining the existing road infrastructure in excellent condition. In these modern times where efficiency and effectiveness is key to maintaining global competitiveness, the Government must ensure that the transport system which includes the road network can deliver the performance that businesses and industries require.

The National Mid-Term Development Plan 2015-2019 of the Government of Indonesia (GoI) allocated ± USD 404 billion for all its infrastructure needs. But the government funding capacity has not been sufficient to finance this infrastructure development fully. It can only provide around 50% of total needs. 20% of the remaining gap is planned to be filled with loans and obligations. This leaves 30% of the funding needs yet to be sourced.

Figure 1. Infrastructure Funding Resources
The Directorate General of Highways (DGH), within the Ministry of Public Works & Housing, is actively engaged in carrying out GoI’s road infrastructure development program in accordance with the Ministry of Public Works and Housing Strategic Plan 2015-2019 prepared to implement its part of the National Mid-Term Development Plan. The main activities in the strategic plan are preservation and development of the national non-toll road network, with the target of 47,017 Km of national roads by 2019 with 98% of it achieving the national standard for road surface stability. This will increase connectivity and accessibility that will enable an improvement in national productivity and competitiveness by 77% over the period.

![DGH Funding Allocation 2012-2017](image)

Figure 2. National Road Network Budget

Some of the increase in the length of the national road network is due to the building of new roads, and much of the increase is the result of road reclassification from regional roads to national roads. Reclassification is determined at a review every five years. Roads that used for national-scale connectivity and roads that support some national development strategy may be reclassified as national roads. The reclassified roads tend to have substandard road surface stability, so the GoI needs to allocate substantial funds for upgrading them to achieve the national target of road surface stability.

To fund all its obligations, DGH must innovate to attract funding resources other than the state budget and loans and obligations. One option is to attract private sector funding through a form of Public-Private Partnership (PPP), which GoI encourages for investment in infrastructure.

2 CURRENT CONDITION

Recent budget cuts have forced DGH to optimise its budget spending and also to examine alternative financing options more urgently. The funds indicated for DGH in the 2015-2019 mid-term development plan was ± USD 20 billion, but in reality, allocations have been far below this which has resulted in a funding backlog of ± USD 4 billion in 2018, estimated to be the same in 2019.
One of the alternatives for filling the backlog to assure that DGH plays its part in strengthening regional and national economic growth is through developing PPP Availability Payment (PPP-AP) scheme. This represents a significant reform in Indonesia’s financial system. Through PPP-AP, the GoI hopes that the road network will be more efficient by increasing service levels while reducing current budget commitments. The use of PPP-AP will require changes in organisational policies, procurement systems and mechanisms, quality assurance procedures and involvement of third parties.

3 DESCRIPTION OF PPP-AP

PPP-AP is one form of partnership between the Government and the Private Sector. It provides for a periodical payment from a government entity to a private entity in exchange for making available to the public infrastructure services that match the qualities and criteria stipulated in a Service Level Agreement (SLA) between government and private entity, as required by Minister of Finance Regulation PMK/190/2015. Failure to provide the level of services stipulated in the SLA is penalised by a reduction in the periodic payments. Service is defined by the quality of the road over the road length and the concession period. The purposes of adopting PPP-AP consist of:
a. An effective solution for accelerating national development;
b. Creating a more sustainable national road service;
c. Increasing the effectiveness of the state budget through providing long-term certainty of the public service output; and
d. Expanding the role of business in the provision of public infrastructure.

Under PPP-AP, the GoI starts paying instalments after completion of construction when the public service provided by the road begins. The construction process will take around two years, with all construction costs provided by the private-sector project company. Instalments continue for a period of 13 years during which the project company must maintain the road to the agreed standard. Total concession period for the project with AP is 15 years.

This can be represented by the formula below:

\[
AP = \text{CAPEX} + \text{OPEX} - \text{Penalty}
\]

- **AP**: Availability Payment
- **CAPEX**: Debt Service Investment Cost, Tax, Rate of Return
- **OPEX**: Operation Cost, Maintenance Cost, Employees
- **Penalty**: Bad Performance Cost

GoI will also provide a guarantee that it will pay the instalments. The government, under the authority of Ministry of Finance, will appoint the Indonesia Infrastructure Guarantee Fund (IIGF), a State-Owned Enterprises, to provide the guarantee. This will assure the project company is paid even if somehow during the payment period the government cannot pay.

Below are the areas of effort needed to implement PPP-AP:

a. Reforming the system for funding road infrastructure, mainly non-toll roads;
b. Forming funding partnerships between a public sector entity and a private sector entity based on the PPP-AP scheme;
c. Allocating risks, responsibilities, and rewards between the public sector entity and the private sector entity; and
d. Effective road infrastructure management involving all related stakeholders:
   - Human Resource Management;
   - Cost Management;
   - Project Management; and
   - Public/Private Organisation Management.
The differences between the conventional way of financing road network services and PPP-AP described in the table below, which illustrates the benefits of PPP-AP.

Table 1. Comparison between Conventional Road Service Provision and PPP-AP

<table>
<thead>
<tr>
<th>Conventional System of Road Service Provision</th>
<th>PPP-AP system of Road Service Provision</th>
</tr>
</thead>
<tbody>
<tr>
<td>Under the conventional system, quality is assured through adequate budgeting every year, with no penalty for poor quality.</td>
<td>Under PPP-AP, quality is assured for the whole period through the Service Level Agreement (SLA) with penalties for non-compliance.</td>
</tr>
<tr>
<td>The rules and standard operating procedures of government agencies limit their ability to mitigate or respond to risks.</td>
<td>Business entities are more able to mitigate and respond to risks better because they are more flexible in budgeting and operating procedures necessary to achieve the required level of service.</td>
</tr>
</tbody>
</table>

The following figure identifies stakeholders and their involvement in PPP-AP for national non-toll road infrastructure development in Indonesia:

Figure 6. Stakeholders in PPP-AP Scheme

There are several criteria used in choosing a National Road to funded using PPP-AP:

a. The national road which is part of the backbone of national economic growth, or national road which is critical for national defence;
b. The national road where high managerial skill and innovation is needed in development and maintenance to achieve high performance and efficiency; and
c. The national road is requiring a sustained, consistent and adequate level of service.
The steps for preparing a PPP-AP scheme illustrated in the graphic below.

Figure 7. AP Scheme

4 RISK FACTORS IN PPP-AP

Risk factors are usually not considered in the conventional payment scheme, which means the overall costs of projects most likely underestimated. Risk factors are reviewed under the PPP-AP scheme, both by the government in estimating the projected funding required in the preliminary study (Step 1 in figure 7) and by the private sector in bidding (as Step 7 in figure 7), so the overall cost to the government is fixed by agreement until the end of the concession period. Risks factors are allocated to the party most competent to handle the risk during the concession period, and for most risk factors this is the private sector partner. An example of an assessment of risk management capabilities shown in the tables below:

Table 2. Mitigation Capability Assumptions

<table>
<thead>
<tr>
<th>No</th>
<th>Risk</th>
<th>Government</th>
<th>Private Sector</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Design Error</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>B</td>
<td>Cost Increase</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>C</td>
<td>Delay</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>D</td>
<td>Overloading</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>E</td>
<td>Construction Quality</td>
<td>1</td>
<td>4</td>
</tr>
</tbody>
</table>

Table 3. The assumption of Risk Allocation

<table>
<thead>
<tr>
<th>No</th>
<th>Risk</th>
<th>Government</th>
<th>Private Sector</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Design Error</td>
<td>0%</td>
<td>100%</td>
</tr>
<tr>
<td>B</td>
<td>Cost Increase</td>
<td>0%</td>
<td>100%</td>
</tr>
<tr>
<td>C</td>
<td>Delay</td>
<td>0%</td>
<td>100%</td>
</tr>
<tr>
<td>D</td>
<td>Overloading</td>
<td>0%</td>
<td>100%</td>
</tr>
<tr>
<td>E</td>
<td>Construction Quality</td>
<td>0%</td>
<td>100%</td>
</tr>
</tbody>
</table>
5 VALUE FOR MONEY

Value for Money (VfM) is a tool used to decide whether the government can earn better value with the private sector producing the service compared to providing the service it on its own.

The elements of VfM are:

a. Risk transfer. The government listing risks that are best transferred to the private sector, and risks which are retained;
b. Output-based specification. Defining service expected from the facility to be provided;
c. Long-term/life-cycle costs. Calculations not only compare construction costs but also costs during the terms of the cooperation agreement;
d. Performance measurement and incentives. Defining performance measures and incentives then outlining measurement procedures;
e. Competition. The value of selecting the best of several offers; and
f. Private sector innovation and management skills. Advantages of private sector managing infrastructure including innovation effectiveness and efficiency in mitigating selected risk compared to government.

VfM uses qualitative analysis and quantitative analysis. The quantitative analysis of VfM is a comparison of the Present Value (PV) of costs of two different solutions to a problem over the years, in this case, a comparison of conventional road development through the Government Budget with the innovative PPP-AP scheme over the planning concession period.

The weaknesses of quantitative VfM analysis is that it requires the input of historical data from similar projects. This limitation can be addressed by conducting FGDs involving experts to reach consensus and discussion with stakeholders on the technical aspects. Such qualitative analysis of VfM has indicated positively that PPP-AP for the procurement of national non-toll road services will be more profitable than the conventional way of procurement. The VfM for the selected pilots are described in the following section.

6 IMPLEMENTATION OF PPP-AP

DGH is preparing the first use of PPP-AP for a non-toll national roads project in Indonesia, and candidate roads were selected for piloting PPP-AP using the filtering criteria mentioned above. The chosen candidate projects for piloting PPP-AP described in the following paragraphs.

![Figure 8. Project Location in Indonesia](image)

**Eastern Sumatera Road Corridor**

a. Eastern Sumatera road corridor is a national road that connects provincial capitals in Sumatera Island. The length of Eastern Sumatera Road Corridor is almost 2,750 Km.
b. This corridor has strategic value, for economic growth by providing ground logistic connectivity, and defence and security of the country. It also connects with the Asian road network.
c. The two selected segments of this corridor pass through marshland and hilly terrain. DGH’s private partner will need to provide specialised treatment in its construction and maintenance to assure that it mitigates risks.
d. DGH’s VfM assessment was positive. The Riau section provides VfM ± USD 6.4 million greater, and South Sumatra section provides VfM ± USD 7.2 million greater than with conventional payment.

Since this corridor serves a crucial national logistics function and carries heavy vehicles, road preservation improvements are urgently required, needing considerable technical expertise in construction and operations, which DGH believes is best obtained by the involvement of the private sector with risks allocated to them as the party that has the capability to overcome them.

**Figure 9. Project Location of Eastern Sumatera Road Corridor**

**Trans Papua Road Corridor Construction**

The Trans Papua Corridor is 3,260 Km long, and a segment 284 km long or 9% of the entire corridor is to be funded by PPP-AP. Around 85% of this segment is not yet paved. The conditions for road construction in Papua are quite complicated because of the limited availability of resources, difficult project logistics and high political profile of road development there. Active and efficient participation of the private sector is needed to construct and maintain this critical section of the Trans Papua Corridor, allowing the government to focus more on other development priorities and security in Papua. VfM for the Trans Papua Road Corridor using PPP-AP is giving a positive value, estimated at more than USD 15 million over the conventional funding scheme.

**Figure 10. Project Location of Trans Papua**
Current Status

At the time of submitting this paper (early June 2018), the Eastern Sumatra Road Corridor PPP-AP packages are at the Final Business Case and Pre-Qualification stages (steps 5 and 6 in Figure 11. Milestones for PPP-AP Implementation and The Trans Papua Corridor PPP-AP package is at the Outline Business Case stage (step 3 in the same figure).

The following are the schedule milestones for the implementation of the PPP-AP scheme:

![Milestones for PPP-AP Implementation](image)

6 IMPACTS OF USING PPP-AP

The implementation of PPP-AP for road infrastructure will create changes for the parties to the agreement and community. The more that PPP-AP is used, the more significant the impact will be. At this moment we expect the following results:

a. The state budget will be used more effectively and efficiently.

The government has had difficulty preserving the road network and achieving a higher level of road stability because of the limitation of its funding. When PPP-AP is successfully implemented, the government will gain increasing benefit because PPP-AP is considered more efficient and effective than the conventional funding. Also, risks which burdened the government before can now be allocated to its private sector partners. PPP-AP is a practical solution for increasing capital expenditure on road infrastructure thus accelerating national development without waiting for the availability of state budget. It also assures good VfM, as the private sector are motivated to secure road infrastructures in excellent condition and a high level of stability.

b. The private sector will have an increased role in national development.

The implementation of PPP-AP will impact institutional arrangements in road network management. In the conventional system, the government involves many parties in project activities (to plan, execute, and oversee contracts). In PPP-AP the government gives private sector entities the role of planning, implementing, and monitoring projects, enabling the government to focus on “steering not rowing”.
c. PPP-AP will become the benchmark for the quality of the national road network. PPP-AP agreements are based on the quality of the service provided to the public with penalties for failing to achieve the prescribed condition. This will set the standard for developing and preserving the whole road network.

d. The community will benefit from higher quality and demand it of the whole network. The community will benefit from having excellent road conditions, reducing the cost of logistics and vehicle operation, and providing safer driving conditions.

e. Local and provincial governments will be motivated to start using PPP-AP. Regional governments generally have more limited capacity to fund capital expenditure in their road networks, that is, they have a relatively greater funding gap for providing road infrastructure. National government success in implementing PPP-AP will motivate a trend within regional governments. Some local governments with high priority for improving their road network are likely to quickly follow the pattern of successful use of PPP-AP for national roads.

Table 4. PPP-AP Scheme Implementation Targets

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Conventional Payment Scheme</th>
<th>PPP – AP Payment Scheme</th>
</tr>
</thead>
<tbody>
<tr>
<td>Capital Cost</td>
<td>Out of the annual budget</td>
<td>Payment in advance by the private sector, then the government pay the instalment</td>
</tr>
<tr>
<td>Life-cycle cost</td>
<td>Likely to be more expensive since the government, less able to respond to risks, bears all risks</td>
<td>Cheaper because certain risks are shared with the private sector which is more able to respond to them</td>
</tr>
<tr>
<td>Implementing Organisations</td>
<td>The public-sector implementer of infrastructure services has full responsibility for producing the infrastructure, from design and construction to ongoing maintenance</td>
<td>The implementer only had a role as supervisor during construction and maintenance throughout the concession period</td>
</tr>
<tr>
<td>Work Quality</td>
<td>Follow the regulation for services standard, but in reality, the quality often produced lower than standard because of no punishment attached</td>
<td>The higher quality delivered since there are an SLA and penalty system for the payment. So private sector need to provide infrastructure services which meet the SLA standards.</td>
</tr>
</tbody>
</table>

In reality, it is not easy to change our ways of providing infrastructure from the existing conventional one to the new approach. There are some obstacles in applying the PPP-AP:

a. Mindsets
Some people think conventionally and are closed to new things, making it hard to implement this new approach. In that matter, socialisation is needed to explain the benefits of change to the public.

b. There is not yet a Standard Operating Procedures (SOP) and SLA standards for PPP-AP. PPP-AP needs detail guidelines for preparing SLA and managing PPP-AP. DGH intends to develop these from early experience with PPP-AP, but in the meantime, all parties must innovate carefully and be reflective at every step.

c. Limited experts in PPP.
PPP-AP is still new. There are few experienced people available to help us, and if Indonesia intends to expand PPP-AP, we will need many more people.

In summary, it will take time to synchronise perceptions of the benefit that can be achieved by using PPP-AP. However, it is not impossible if we can help everyone to be open to the concept, prepare. Standard Operating Procedures, and develop the full legal basis for making and managing the PPP-AP SLA mechanism in detail.
7 FUTURE PROSPECTS OF PPP-AP

The future infrastructure services in Indonesia is expected to be provided increasingly with private sector investment so that private sector will frequently play a significant role in road network services. The government will become less involved in construction and more in policy oversight, as indicated as Stage 4 in the following figure that shows the chronology of approaches to infrastructure provision of the Ministry of Public Works and Housing over time.

Figure 12. Infrastructure Provision Chronology over Time

Increasing of private sector involvement is expected to improve cost efficiency and decrease unit cost of logistics so that economic growth in the region can increase.

Figure 13. Illustration Level of (i) Cost-Effectiveness over Time and (ii) Unit Cost of Logistics over Time
CONCLUSION

The limited budget for providing road infrastructure is an obstacle in the way of the government in providing a level of road infrastructure for people that provides safe and unencumbered connectivity that will encourage economic development, while the needs for road infrastructure always increase in every year. To fill the funding gap, the government must continuously innovate in funding, for example, by using the PPP-AP. Using PPP-AP is a new concept for providing road infrastructure in Indonesia. Ministry of Public Works and Housing through the Directorate General of Highways is currently preparing this scheme for sections of the Eastern Sumatera Road Corridor and the Trans-Papua Corridor. Many benefits will be gained when this scheme is implemented successfully:

a. Costs: PPP-AP can reduce the cost of providing infrastructure because risks are bestowed upon the private sector more able to mitigate them.

b. DGH focus: As implementer DGH will move from producer to provider and SLA supervisor and can focus on policy and other projects.

c. Quality of works: Quality is assured through provisions of the SLA.

However, all stakeholder parties need to contribute, be open to change, minded and be active in applying PPP-AP so that implementation of this scheme can reach its expected targets.

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The government of Indonesia's Regulation No. 34/2006 on Road.
Minister of Finance Regulation No. 8/2016 (Amendment to MoF Regulation No. 260/2010).
Minister of Finance Regulation No. 190/2015 on Availability Payment in connection with PPP for the Provision of Infrastructures (Availability Payment).
Presidential Regulation No. 38/2015 on Public Private Partnership in the Provision of Infrastructures.
Presidential Regulation No. 78/2010 on Infrastructure Guarantee in PPP Projects by Infrastructure Guarantee Company.
<table>
<thead>
<tr>
<th>PAPER TITLE</th>
<th>Rural Road Selection Factor Analysis to Support Multimodal Transport After Completion of Double-Track Rail in the Northeast of Thailand</th>
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</thead>
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<td>AUTHOR</td>
<td>TRACK</td>
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<tr>
<td>(Capitalize Family Name)</td>
<td>POSITION</td>
</tr>
<tr>
<td>Koson JANMONTA</td>
<td>Civil Engineer</td>
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<td>Wit RATANACHOT</td>
<td>Chief Engineer</td>
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<tr>
<td>Koonnamas PUNTHUTAECHA</td>
<td>Civil Engineer</td>
</tr>
<tr>
<td>Kitti MANOKHOON</td>
<td>Civil Engineer</td>
</tr>
<tr>
<td>E-MAIL (for correspondence)</td>
<td><a href="mailto:Koson.te@gmail.com">Koson.te@gmail.com</a></td>
</tr>
</tbody>
</table>

KEYWORDS: Double-track rail, exploratory factor analysis, multimode transportation, rural road.

ABSTRACT: Thailand is one of the countries with a fast growing economic therefore to be able to save the logistics cost makes Thailand a competitive country in the Asian region. With the upcoming double-track rail project, it is expected that the use of multimode transportation, especially road and train modes, will increase and lead to significant cost saving in logistics. As rural roads are key roads to connect the supply chain, from the product source to the end customer, it is necessary to select crucial rural roads to support the double-track rail transportation. This study aims at identifying key factors affecting rural road selection to support the road and double-track rail transportation in the Northeast of Thailand utilizing an exploratory factor analysis method. The results reveal four key factors which are 1) Product, 2) Product Source, 3) Road Network, and 4) Traffic Volume factors, with a total of 18 associated sub-factors. The four factors are confirmed with the reliability test, with high alpha values. It is expected that the study results would help the Department of Rural Roads for better understanding and indicating the right key roads to support multimode transportation after completion of double-track rail in the future.
Rural Road Selection Factor Analysis to support multimodal transport after completion of double-track rail in the Northeast of Thailand

Koson Janmonta¹, Wit Ratanachot¹, Koonnamas Punthutaecha¹, Kitti Manokhoon¹
¹ Department of Rural Roads, Ministry of Transport, Thailand

1. INTRODUCTION

Thai economy has been significantly growing in the past years from agricultural, industrial and services sectors. The development of infrastructure and transportation system is, therefore, crucial for supporting the economic growth. Recently, Thai government has promoted multimodal transportations to increase efficiency and effective of transportation system and reduce logistics cost. One of the major plans is the double-track railway project for logistics cost and link the transportation throughout the region.

The double-track rail program (phase I) consists of six routes, in which two of them are approved and under construction, including 1) Chachoengsao – Klong 19 – Kangkoy and 2) Chira Station – Khonkaen. Moreover, the Chira Station – Khonkaen route will be expanded to Nongkai and across to Laos in the second phase, making it convenient to transport the products to the region (Panchai 2017). The Chira Station - Khonkaen route has three container yards (CY), two in Nakorn Ratchasima and one in Khonkaen. These container yards serve as the transferred link between the truck and train modes. This is beneficial, especially for the agricultural products that require large space for the transferring process (Office of Transport and Traffic Policy and Planning 2017). However, it is found that the road network to connect the sources, factories, distribution points, and container yards are yet effective, especially rural roads that are not designed to accommodate high load and volume traffic. This, in turn, deteriorates the rural roads, and causes high maintenance (Department of Rural Roads 2016).

This paper, therefore, aims at identifying factors affecting rural road selection to support the road and double-track rail transportation in the Northeast of Thailand, utilizing the questionnaire survey and exploratory factor analysis (EFA) method. It is expected that the Department of Rural Roads (DRR) use the study results to better understand key factors affecting rural road selection to support multimode transportation, and better plan for road maintenance.

2. RAILWAY TRANSPORTATION IN THAILAND: AGRICULTURAL AND INDUSTRIAL PRODUCTS

Polyiam (2009) stated that the development of multimode transportation, especially truck and train modes, assists in reducing the logistics cost, and increasing the opportunities to trade with neighboring countries. However, the transportation of agricultural and industrial products in Thailand still depends mainly on the truck mode though the logistics is higher than the rail mode (Termpittayapaisit, 2011). To explain, the truck mode consists of 87.5% of the total transportation, while the train and water modes are accounted for
only 11.08% and 1.40% of the total transportation, respectively. The logistics cost per ton-kilometer for the truck mode is, however, more than two times of those in train and water modes (Office of the National Economic and Social Development Board, 2016).

Many government plans are, therefore, initiated to promote the use of multimode transportation. For example, Ministry of Transport (2015) initiated the double-track rail program to promote the use of rail mode and expected that it will help double the use of the rail mode and reduce the logistics cost per gross domestic product (GDP) by 2%. The Northeast Economic and Social Office (2013) mentioned that the Northeast of Thailand is the source of major agricultural products and can be promoted to be a hub in the Asian by improving the multimode transportation, especially those between truck and train modes.

A number of studies have also been conducted in the area of logistics and multimode transportation. The Department of Rural Roads (2013), for example, initiated the feeder system to connect the main and rural roads across the country to accommodate the logistics and transportation in Thailand. Five key factors were used to select the appropriate roads, including 1) linkage and connectivity, 2) traffic volume, 3) environmental issue, 4) engineering issue, and 5) cost. Based on the analytic hierarchy process results, it was concluded that the linkage and connectivity factor is the most important factor, while the cost factor is the least important factor. The Department of Rural Roads (2016) also studied the possibility of using multimode transportation for rice transportation in the North and Central parts of Thailand utilizing a structural equation modeling technique. The study results revealed the necessity to improve the rural road standard to serve with the multimode transportation in the future with five key factors, namely 1) road network, 2) road geometry, 3) multimode transportation, 4) product source, and 5) traffic volume.

Department of Agriculture (2013) stated that key agricultural products in Thailand that can be transported using the multimode are rice, sugar cane, cassava, and rubber. Ministry of Industry (2016), on the other hand, mentioned that the industrial products, including cement, stone, and potash, can be transported with multimode with lower logistics cost. Chanchaipetch and Kritchanchai (2009) examined the use of multimode for rubber transportation in Thailand, and concluded that rubber can be transported using truck, train, and ship modes. Udonthani Chamber (2015), on the other hand, mentioned that potash can be exported using truck and train modes.

Based on the above literatures, this study lists a total of 18 items affecting rural road selection to support the road and double-track rail transportation in the Northeast of Thailand, as shown in Table 1. These items are used for questionnaire survey development to collect data for the exploratory factor analysis.
Table 1. Items affecting rural road selection to support the road and double-track rail transportation in the Northeast of Thailand

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Main road connectivity</td>
<td>Polyiam 2009, Panchai 2017</td>
</tr>
<tr>
<td>2</td>
<td>Rural road network</td>
<td>Department of Rural Roads 2013, Department of Rural Roads 2016</td>
</tr>
<tr>
<td>3</td>
<td>Community area</td>
<td>Department of Rural Roads 2013, Department of Rural Roads 2016</td>
</tr>
<tr>
<td>4</td>
<td>Road geometry</td>
<td>Department of Rural Roads 2013, Department of Rural Roads 2016</td>
</tr>
<tr>
<td>5</td>
<td>Train linkage point</td>
<td>Termpittayapaisit 2011, Panchai 2017</td>
</tr>
<tr>
<td>6</td>
<td>Rice source</td>
<td>Department of Agriculture 2013, Department of Rural Roads 2016</td>
</tr>
<tr>
<td>7</td>
<td>Rubber source</td>
<td>Chanchaipetch and Kritchanchai 2009, Department of Rural Roads 2016</td>
</tr>
<tr>
<td>8</td>
<td>Sugar cane source</td>
<td>Department of Agriculture 2013, Department of Rural Roads 2016</td>
</tr>
<tr>
<td>9</td>
<td>Cassava source</td>
<td>Department of Agriculture 2013, Department of Rural Roads 2016</td>
</tr>
<tr>
<td>10</td>
<td>Rice processed area</td>
<td>Department of Agriculture 2013, Department of Rural Roads 2016</td>
</tr>
<tr>
<td>11</td>
<td>Rubber processed area</td>
<td>Chanchaipetch and Kritchanchai 2009, Department of Rural Roads 2016</td>
</tr>
<tr>
<td>12</td>
<td>Sugar cane processed area</td>
<td>Department of Agriculture 2013, Department of Rural Roads 2016</td>
</tr>
<tr>
<td>13</td>
<td>Cassava processed area</td>
<td>Department of Agriculture 2013, Department of Rural Roads 2016</td>
</tr>
<tr>
<td>14</td>
<td>Stone processed area</td>
<td>Department of Agriculture 2013, Department of Rural Roads 2016</td>
</tr>
<tr>
<td>15</td>
<td>Cement processed area</td>
<td>Department of Agriculture 2013, Department of Rural Roads 2016</td>
</tr>
<tr>
<td>16</td>
<td>Potash area</td>
<td>Udonthani Chamber 2015, Department of Rural Roads 2016</td>
</tr>
<tr>
<td>17</td>
<td>Truck volume</td>
<td>Department of Agriculture 2013, Department of Rural Roads 2016</td>
</tr>
<tr>
<td>18</td>
<td>Traffic volume</td>
<td>Department of Agriculture 2013, Department of Rural Roads 2016</td>
</tr>
</tbody>
</table>

3. QUESTIONNAIRE SURVEY AND DATA SCREENING

3.1 Questionnaire survey and data collection

The above 18 items are used to develop a questionnaire survey to collect data. The survey consists of two parts. Part I is devoted to gathering demographical information about the respondents and their respective organizations to ensure that the respondents have appropriate backgrounds. Part II covers 18 statements asking the respondents to rate their agreement with each item affecting rural road selection to support the road and double-track rail transportation in the Northeast of Thailand. The 5-point Likert scale is used, with point 1 representing ‘strongly disagree’ and point 5 representing ‘strongly agree’. Examples of statements are “The ability to connect the rural roads with main roads is a criterion for rural road selection to support multimode transportation” and “Road geometry is a criterion for rural road selection to support multimode transportation”.

Target respondents in this study are government officials and managers in logistics- and transportation-related organizations involved in various transportation decision-makings in Nakorn Ratchasima, Khonkaen, Udonthani, and Nongkhai provinces in Thailand. The designated areas are based on the route along the double-track rail transportation in the Northeast of Thailand. One hundred and twenty
questionnaires are sent, with 81 returns, representing a 67.80% response rate. Almost half of the respondents come from Khonkaen and Nongkhai provinces, respectively, as shown in Figure 1.

Figure 1. Province of the respondents

Around half the respondent’s work in the Department of Rural Roads and local authorities, respectively (see Figure 2). Moreover, most of the respondents are in management positions, and have at least five-year working experience (see Figures 3 and 4). These prove the suitability of the respondents to provide the information for the analyses.

Figure 2. Organization of the respondents
3.2 Data screening

The collected data are screened to increase the confidence of the data. Two data screening processes are used in this study, including the normality and outlier tests. Two important components of normality are skewness and kurtosis (Tabachnick and Fidell 2007). Skewness relates to the symmetry of the distribution, while kurtosis, on the other hand, relates to the peakedness of a distribution. When a distribution is normal, the value of skewness is zero and that of kurtosis is three (Pallant 2005). According to Curran, West, and Finch (1996), the values of skewness and kurtosis should not be more than 2 and 7, respectively, to explain the normal distribution of the data. The results demonstrate that all 18 items show normal distribution, thus increasing confidence in the data.

The box plot is, on the other hand, performed to detect outliers. The results show that data numbers 70 and 74 show signs of outliers (see Figure 5). As a result, these two data sets are removed, resulting in a total of 79 data sets for the EFA.
4. **EXPLORATORY FACTOR ANALYSIS**

In this study, an exploratory factor analysis is performed to explore relationships among items and facilitate construct formulation using the Statistical Package for Social Sciences (SPSS) software. The SPSS software is a Windows based program that can be used to perform data entry and analysis and create tables and graphs (The University of Vermont, 2016). It is capable of handling large amounts of data and is commonly used in social sciences studies. It consists of the following steps:

- Set a problem statement;
- Extract the related items from a number of literature reviews;
- Select the extraction method, such as the principal component analysis (PCA), generalized least square, and principal axis factoring methods;
- Select factor rotation method, such as the quartimax, equamax, and varimax rotation methods;
- Select factor loading value;
- Perform the Kaiser-Meyer-Olkin (KMO) and Bartlett’s Test of Sphericity to confirm the use of the EFA (Tabachnick and Fidell 2007); and
- Perform the analysis and name the extracted factors.

In this study, the principle component analysis method, together with varimax rotation and factor loading of 0.3 are used to extract the 18 items into key factor affecting rural road selection to support the road and double-track rail transportation in the Northeast of Thailand (Tabachnick and Fidell, 2007). The analysis results group 18 items into four key factors, as shown in Table 2.
Table 2. The EFA results

<table>
<thead>
<tr>
<th>Item</th>
<th>Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Product</td>
</tr>
<tr>
<td>Cement processed area</td>
<td>0.89</td>
</tr>
<tr>
<td>Potash area</td>
<td>0.86</td>
</tr>
<tr>
<td>Stone processed area</td>
<td>0.82</td>
</tr>
<tr>
<td>Rice processed area</td>
<td>0.75</td>
</tr>
<tr>
<td>Rubber processed area</td>
<td>0.74</td>
</tr>
<tr>
<td>Cassava processed area</td>
<td>0.65</td>
</tr>
<tr>
<td>Sugar cane processed area</td>
<td>0.58</td>
</tr>
<tr>
<td>Rice source</td>
<td></td>
</tr>
<tr>
<td>Sugar cane source</td>
<td></td>
</tr>
<tr>
<td>Cassava source</td>
<td></td>
</tr>
<tr>
<td>Rubber source</td>
<td></td>
</tr>
<tr>
<td>Community area</td>
<td></td>
</tr>
<tr>
<td>Train linkage point</td>
<td></td>
</tr>
<tr>
<td>Main road connectivity</td>
<td></td>
</tr>
<tr>
<td>Rural road network</td>
<td></td>
</tr>
<tr>
<td>Road geometry</td>
<td></td>
</tr>
<tr>
<td>Truck volume</td>
<td></td>
</tr>
<tr>
<td>Traffic volume</td>
<td></td>
</tr>
</tbody>
</table>

Factor 1 consists of seven items explaining mainly about the product, so it is called the Product factor. Factor 2, on the other hand, consists of five items considering mainly about the product source; this factor is thus called the Product Source factor. Four items, including the train linkage point, main road connectivity, rural road network, and road geometry, are grouped together, and called the Road Network factor. The last factor, Factor 4, consists of two items about the traffic volume, and is called the Traffic Volume factor.

The four extracted factors are then performed with the reliability test to confirm the grouping of the four factors.

5. THE RELIABILITY TEST

To increase confidence in the factors extracted from the EFA, a reliability test is performed using a measure of internal consistency (Cronbach’s alpha). In a good solution, Cronbach’s alpha (α) ranges between zero and one; the larger the value, the more stable the factors. A high value means that the observed variables account for substantial variance in the factor scores, while a low value means the factors are poorly defined by the observed variables. According to Sharma (1996), alpha values ranging from 0.6 to 0.7 are regarded as sufficient, and a value higher than 0.7 is regarded as good, in reliability testing.
In this study, the four factors are confirmed with the reliability test. The results have alpha values ranging from 0.75 to 0.93 (see Table 3), all of which were considered highly reliable. This thus increases confidence in the contribution of the 18 items to the measurement of their respective constructs.

**Table 3.** The reliability test results

<table>
<thead>
<tr>
<th>Factor</th>
<th>Cronbach’s alpha value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Product</td>
<td>0.90</td>
</tr>
<tr>
<td>Product Source</td>
<td>0.93</td>
</tr>
<tr>
<td>Road Network</td>
<td>0.75</td>
</tr>
<tr>
<td>Traffic Volume</td>
<td>0.86</td>
</tr>
</tbody>
</table>

6. **CONCLUSION**

Department of Rural Roads has one the main responsibility is to maintain safe road condition. With the upcoming double-track rail system, the department needs to effectively plan for road improvement to support more of product transportation in the future. This study utilizes the exploratory factor analysis to select rural road to support the multimode transportation in the Northeast of Thailand. A total of 18 items are extracted from the literature reviews and are used for a questionnaire survey development to collect data for the analysis. The analysis results extract four key factors affecting rural road selection to support the double-track rail transportation in the Northeast of Thailand, namely 1) Product, 2) Product Source, 3) Road Network, and 4) Traffic Volume factors. it is expected that the Department of Rural Roads use the study results to better understand and select proper roads to support the road and double-track rail transportation in the future.
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PAPER TITLE: Numerical Model of Reinforced-Concrete Moment Resisting Frames and Effect of Masonry Infill walls in Progressive Deterioration

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KEYWORDS: progressive deterioration; infill walls; crack development; displacement; reinforced concrete

ABSTRACT:
Structural collapse under the sudden failure of structural member is a curtail issue in concrete structures. However, non-structural members such as masonry infill wall reduces the progressive deterioration caused by column failure. Masonry infill-wall influences the degree of damage due to crack propagation, frame displacement, and adjacent beams rotation in reinforced concrete (RC) structural analysis. The aim of this research is to analyze the effect of masonry infill wall to assess the failure pattern of the adjacent concrete columns in RC moment resisting frames. This numerical analysis measured and compared the behavior of the five RC frame models with masonry infill wall subjected to the progressive failure. The crack propagation behavior, ultimate strength, displacement, failure mechanism, rebar load capacity, beam rotation after collapsing the adjacent column were investigated. The results indicated masonry infill walls transferred the applied load within other structural members. Masonry infill walls played substantial role in decreasing the vertical displacement of RC frames when progressive deterioration occurred. Therefore, RC frames ultimate strength was enhanced and failure mechanism was postponed. However, the RC frames flexibility reduced and brittle failure happened. The findings showed masonry wall location is an important contributor to reduce the displacement and degree of flexibility of the RC structures. Consequently, the impact of masonry infill walls was more effective in the second floor and some of first floor beams.
Modelling Reinforced-Concrete Moment Resisting Frames and Effect of Masonry Infill Walls in Progressive Deterioration

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ABSTRACT

Structural collapse under the sudden failure of structural member is a curtail issue in concrete structures. However, non-structural members such as masonry infill wall reduces the progressive deterioration caused by column failure. Masonry infill-wall influences the degree of damage due to crack propagation, frame displacement, and adjacent beams rotation in reinforced concrete (RC) structural analysis. The aim of this research is to analyze the effect of masonry infill wall to assess the failure pattern of the adjacent concrete columns in RC moment resisting frames. This numerical analysis measured and compared the behavior of the five RC frame models with masonry infill wall subjected to the progressive failure. The crack propagation behavior, ultimate strength, displacement, failure mechanism, rebar load capacity, beam rotation after collapsing the adjacent column were investigated. The results indicated masonry infill walls transferred the applied load within other structural members. Masonry infill walls played substantial role in decreasing the vertical displacement of RC frames when progressive deterioration occurred. Therefore, RC frames ultimate strength was enhanced and failure mechanism was postponed. However, the RC frames flexibility reduced and brittle failure happened. The findings showed masonry wall location is an important contributor to reduce the displacement and degree of flexibility of the RC structures. Consequently, the impact of masonry infill walls was more effective in the second floor and some of first floor beams.

1 INTRODUCTION

Infill masonry walls are considered as non-structural element or partition (M.H. Tsai & T.C. Huang 2010). However, the effectiveness of infill masonry walls in hazardous seismic zones is important when the RC structure is subjected to the lateral loads (Akin LA. 2006). Progressive failure is the main reason of common collapse in reinforced concrete structures. After failure of a structural member such as column, progressive failure begin developing until the collapse of the structure (B.R. Ellingwood & E.V. Leyendecker 1978). The main structural members that play important role in transferring loads are columns. Therefore, the failure of RC columns are crucial issue to be investigated (A. Cachado et al. 2011). Infill masonry walls introduced as an alternative path for crack development and postponement of the RC progressive failure. Furthermore, the infill masonry walls increased the safety with transforming structural member’s sudden failure to expected failure mode (S. Farazman, B.A. Izzuddin &D. Cormie 2013). Additionally, Infill masonry walls improved the behavior of RC structures subjected to the lateral loads. The RC structural failure is due to brittleness of cement. However, infill masonry walls mechanism allowed the structure to continue its life service while soft-story mechanism begun. Additionally, it increased the stiffness of the RC structures (G. Mondal, S. & Tesfamariam 2013).

2 ANALYSIS OF MASONRY INFILL WALLS IN RC FRAMES

Three dimensional models are widely used to determine the 3D effect in reinforced concrete structures. When the effect of the 3D models are not taken into account, the results of the two-dimensional models is conservative (Maheri Mahmoud R & Pourfallah S. 2012). However, due to the fact that the behaviour of the progressive failures is mainly due to the response of the frame, the 2D dimensional model has satisfactory accuracy. The analysis also reflects the effect of masonry infill walls to postpone the progressive failure and increase the ultimate RC frame capacity. Therefore, the 2D method analysis was applied in this numerical analysis.

3 REINFORCED CONCRETE FRAMES ANALYSIS

The results of numerical analysis for five reinforced-concrete frames with masonry infill walls indicated the variation in strength, displacement, strain, crack development, and rebar failure. The findings demonstrated that the progressive failure for all models were categorized into three main stages including initial stage, compression stage, and
chain changes stage. The main contributor in postponing progressive failure is the rebar in the beam compression zone which is related to strain transformation from compression to tension region.

It is important that the beam deformation begun as deflection was increasing during these stages. The displacements of the adjacent columns toward the moving direction (horizontal displacement of 0 mm). In the Figure 1-4, the displacement of all four frames was compared with control frame (Bare Frame). “M” is acronym for each frame model and the number presents 1-4 different frame model (M1-M4). M4 has the greatest resistance force of 140 Kn with the smallest vertical deflection of approximately 20 mm (Figure 1).

4 REINFORCED CONCRETE FRAME M1 ANALYSIS

Figure 1 shows the vertical displacement force for central concrete column without masonry infill wall which was called bare frame. Resistant force at the compression stage of 109.8 mm in vertical displacement reached to the maximum of 32.29 Kn. Then, it gradually increased to 39.24 (up to a displacement of 325.41 mm) at the end of the compression stage. At this stage, the development of maximum deflection was revealed in the damaged structure and the resistance strength was increased again in the maximum deflection until the end of the test. However, in some stage, this force slightly reduced due to the breakdown of the meshes. The vertical displacement for frame M1 reached to 325.41 mm.

Figure 2 shows the vertical displacement under the central concrete column. The failure of central column and impacted areas in the adjacent beams is shown in the Bare Frame (without infill walls). The displacement along central column increased until the yield of structural column was occurred. Consequently, the connected beams were affected with high displacement in the column-beam connection.
4.1 BARE MODEL ANALYSIS

The numerical analysis indicated the vertical displacement is 37.5-53.29 mm because of crack propagation within the masonry infill wall in the reinforced concrete frame which caused notable reduction in the resistance force. After this stage of failure, the resistance force increased again in the model and remained constant until the ultimate load was applied. The maximum resistance force reached to 37.52 mm at the vertical displacement of 41.31 mm.

When the resistance force was reduced, the displacement simultaneously had a slight increase due to deflection of the beam at the 41.65 kN ultimate load. However, the displacement data was fluctuated during applying the load at this stage especially where the deflection was maximum. This reduction may occurs due to rebar failure. The test was completed when the vertical displacement reached to 316.4 mm.

Figure 3 indicates the vertical displacement and the failure of the concrete frame in M1(infill walls in the adjacent spans). The rebar stress reached to more than yielding stress. The major difference between the non-masonry models is that the concrete frames in both side remained horizontally. Additionally the maximum tension in the model meshes with the masonry infill wall was 488 MPa, while the tension in the model without masonry walls was close to the ultimate stress of 516 MPa. Furthermore, the amount of resistance force at the beginning of the failure mode in this model with the masonry infill wall is 58% higher than the non-masonry infill wall model.

![Figure 3. Crack development and failure pattern for M1 model (infill walls in the adjacent spans)](image)

4.2 MODEL 2 ANALYSIS

The second model had infill walls in the same span as the central column was located. In M2 model, the resistance force against displacement of central column with masonry infill wall is given in Figure 4. The vertical displacement of the central column reached to 29.45 mm and vertical load reached to 95.3 Kn. Immediately, after failure of the central concrete column, the cracks were developed in the masonry infill wall. Concurrently, the members of the concrete beam were cracked with dropping in the force resistance. In this model, no resistance increase occurred after the failure of the concrete beams.

In Figure 4, the crack propagation increased by the factor of 1.5 in the adjacent masonry infill walls. Despite the relatively high strength of the M2 model compare to M1 model it showed a 25% decrease in resistance at ultimate applied load. In this model, the concrete frame collapsed after failure of masonry infill wall because of the vertical displacement in the middle of the beam. After failure of masonry infill walls, cracks due to tension stress exceeded the maximum allowable stress in the rebar in beam-to-column connection. In this scenario, the theoretical plastic hinge was created.
4.3 MODEL 3 ANALYSIS

The vertical displaced force for a frame with masonry infill wall (M3) with four masonry infill walls on the second floor is also shown in Figure 1. The vertical displacement of the central column is 24.45 mm, the maximum load that the concrete frame carried is 130 kN. Soon after the central column was failed, the cracks were rapidly developed in the masonry infill walls, adjacent walls, and the concrete members. Upon removal of central column, the resistance force considerably decreased.

In M3 model, resistance force reduction was severe compared to other models. The reduction was estimated approximately 26%. When the concrete frame had 50 mm displacement, gradually resistance decreased. In this model, the beams and side columns on the second floor remained safe without any major damage, but the beam in the first floor collapsed. As can be seen in Figure 5, the cracks diagonally developed in middle of the wall.

4.4 MODEL 4 ANALYSIS

The vertical displaced force for a frame with masonry infill wall (M4) with four masonry infill walls on the second and two infill walls in the floor is shown in Figure 7. The vertical displacement of the central column is 23.45 mm, and the maximum load that the concrete frame carried is 139 kN. Soon after the central column was failed, the cracks were rapidly developed in the masonry infill walls, adjacent walls, and the other concrete members. Upon failure of central column, the resistance force considerably decreased.

M4 model compared to Model 3 had less reduction in resistance force. When the concrete frame had 50 mm displacement, frame resistance remained constant. Then, it gradually decreased. In this model, the beams and side columns on the second floor remained safe without any major damage, but the beam in the first floor collapsed.
5 RESULTS OF ANALYZING DISPLACEMENT AND VERTICAL ROTATION

Figure 7 shows the vertical displacement graph verses rotation of the beam. The behaviour of the concrete frame without masonry infill wall is slightly different than the concrete frames with masonry infill walls in the side spans. However, in models M2, M3, and M4, there is significantly different in vertical displacement and beam rotation angle. In M2 and M3, the rotation angel was small. Therefore, these model revealed greater force resistance. Additionally, the correlation between vertical displacement and rotation angle in M2 and M3 was nonlinear while in M4 this correlation was linear.

The vertical displaced force for a frame with masonry infill wall (M4) with four masonry infill walls on the first and second floor is also shown in Figure 7. The vertical displacement of the central column is 23.45 mm, the maximum load that the concrete frame carried is 139 kN. Soon after the central column was failed, the cracks were rapidly developed in the masonry infill walls, adjacent walls, and the concrete members also cracked. Upon failure of central column, the resistance force considerably decreased. M4 model compared to Model 3 had less reduction in resistance force. When the concrete frame had 50 mm displacement, frame resistance remained constant. Then, it gradually decreased. In this model beams and side columns on the second floor remained safe without any major damage, but the beam in the first floor collapsed.

According to the data presented in Figure 7, when the vertical displacement increases in the initial stage and compression stage simultaneously with the development of the cracks width in beam-to-column connection with the limited damage in the concrete. The full involvement of the rebar’s in the bearing and re-loading of the load. The virtual plastic hinge was created into columns in the vicinity of the beam connection to the column and reach to the rupture during constant applying the load. The displacement of 12.7 mm was produced with crack penetration in the tensile region of the concrete members. This phenomena continued to shift to 57.14 mm with a low gradient relatively before extensive cracking. The displacement of 57.14 millimetres was practically transferred to tensile rebar.

6 SUMMARY AND RECOMMENDATION FOR FUTURE RESEARCH
In this numerical research, the behavior of reinforced concrete frames with masonry infill walls subjected to progressive failure were assessed and compared. The behavior of concrete crack propagation after collapsing central column was simulated and analyzed. Then the system of forces, vertical displacement, and rotation were investigated after column removal. The correlation between vertical displacement and rotation angle in M2 and M3 was nonlinear while in M4 this correlation was linear. M4 model compared to Model 3 had less reduction in resistance force. This finding indicated the importance of infill walls in the RC resisting frame and postpone the progressive failure.

The results of the computational analysis indicated that loads were transferred from the beams. The masonry infill walls decreased the deformation of reinforced concrete frames under lateral loads and change failure mechanism. However, the masonry infill walls had considerable contribution in postponing progressive failure as well as improving the reinforced concrete strength behavior in sudden failure.

7 RECOMMENDATION FOR FUTURE RESEARCH

Further experimental and numerical research is required to investigate all aspects of the masonry infill wall effect on the reinforced concrete frames. It is recommended that this computational research is conducted with increasing the number of floors with more beam spans. Investigation of masonry infill walls stiffness is one of the key components in the reinforced concrete structures behavior. Additionally, the effect of compression and tensile strength in the concrete during progressive failure should be taken into consideration.

REFERENCES

ABSTRACT:
Concrete is a resilient, sturdy and durable product for construction. Due to its efficiency in construction considering different environmental and cost-effective characteristics. Concrete materials are omnipresent even in this modern era where several other building materials have been discovered for construction. The different functional aspects of concrete are yet to be discovered. Photocatalysis has added another beneficial feature of concrete to its name. Heterogeneous Photocatalysis, This technology has made its way in the industry to this extent that now actual experimentation proves its use in the practical field. This paper discusses a cost-efficient, environmental friendly (self-cleaning) process. The photocatalytic applications for a sustainable environment are worth mentioning. The applications of the paper discusses the application of photocatalytic products in highway engineering for a road environment which will efficiently eliminate the harmful nitrogen oxides (NOx) and volatile organic compounds (VOCs). The dominance of using photocatalysis by utilizing rain water and sunlight has opened new doors for such self-cleaning and self-disinfecting products in the construction industry. This study will try to bolster the fact that a photocatalytic building material can be utilized in the industry without any kind of second thoughts for the project stakeholders. The result will be a safer environment which will reduce the harmful NOx and VOC’s.
Review on Sustainable applications of Photocatalysis

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1 INTRODUCTION

People spend their major amount of time indoors. It is very much obvious that the quality of environment has a notable impact on their health. After the discovery of the photocatalytic process, the examination of photocatalytic materials for betterment of the environment has achieved momentum. Increasing levels of pollution in different parts of the world have awakened researchers about protecting the environment from various pollutants by using sustainable components which support a long term ecological balance. The characteristics and the applications of the photocatalytic materials are undoubtedly a prudent choice for today’s world where pollution is added in a proliferated amount because of vehicle and industry emissions. Fungi and Bacteria contribute towards the decay of building materials which not only influences the aesthetic properties of the building but, also causes adverse health problems. TiO₂ can also be used for anti-bacterial coatings. In the Textile industry, Photocatalysis has demonstrated its importance in textile which are frequently deemed as our second skin. Photocatalysis can also be used for purposes like deterioration of various organic pollutants in waste water. Photocatalysis does not allow conversion of pollutants from one phase into another rather it allows the development of non-toxic products. In the construction industry the demand of pavement increases as the populated areas grow. The photocatalytic materials have also tried to gain control upon the negative effects induced by the pavement. The semi-conductor photocatalyst possesses the potential to finish mineralization of organics without giving rise to any other type of pollution. The photochemistry of TiO₂ has become a critical research subject for the future. It is mandatory for people to start thinking about using solar energy for the safety of our environment. Solar energy is an immense natural resource which has a never ending limit. Different research projects in the world are striving hard for a better, sustainable and green environment for our future generations. This subject gained momentum in research when the photocatalytic splitting of water on TiO₂ and Sr-doped TiO₂ respectively became successful in the 1970’s. Two important effects related to nature of photoactive TiO₂ coatings had by this time been discovered a)- self-cleaning effect due to redox reactions promoted by sunlight (or in general weak U.V. light) on the photocatalytic surface and b)- the photo-induced super hydrophilicity of the catalyst surface, which enhances the self-cleaning effect(inorganics causing dirt and stains on surfaces can be easily removed due to rainwater soaking between the adsorbed substance and the TiO₂ surface). For the photocatalysis to initiate U.V. light is fundamental which means that this process will be more productive in the day time than in any other time. The solar energy reaching the earth’s surface is about 5 x 10¹² Joule per year. This is more than the 10⁹ times of the annual worldwide consumption of energy. This is the best part that we are using natural resources for this process. This enormous source of energy is utilized with exceptionally engineered construction materials which have the potential to make the environment cleaner and free of pollutants.

1.1 PHOTOCATALYSIS

Photocatalysis is the photoreaction in the presence of a catalyst. The term "photocatalysis" is in widespread use and is here to stay; it is not meant to, nor should it ever be used to, imply catalysis by light, but rather the "acceleration of a photoreaction by the presence of a catalyst". The term "photoreaction" is sometimes elaborated on as a "photoinduced" or "photocatalyzed" reaction, all to the same effect (Mills and Hunte, 1997). Photocatalysis involves the utilization of light to activate photocatalyst which alters the rate of chemical reaction without its own involvement. The process of photocatalysis is very prominent in the presence of light as the photocatalytic efficiency of the building materials is visible. The study of photocatalytic reactions was initiated in 1970’s. During the photocatalytic process light is consumed by one or two reacting species and this is the reason that catalyst is added which does not get consumed and accelerates the reaction.

1.2 PHOTOCATALYSIS AND UNITED STATES GREEN BUILDING COUNCIL (USGBC)

The advantages of utilizing photocatalysis have such sustainable applications that the USGBC encourages the construction industry to utilize photocatalytic cementitious materials for maintaining the aesthetic properties of the building structure and for improving the quality of air. In particular, the innovative utilization of a system comprising TiO₂ in acquiring the LEED certification which is offered by the USGBC, LEED points are awarded for the usage of photocatalytic cementitious materials. The products can be used in the inside and the exterior of building. Such as
Photocatalytic concrete can be used in the exterior of the building where as photocatalytic interior paints have also been introduced in the market which ensure an air purifying effect and maintain the indoor air quality which influences the health of people in a positive way. According to different researches, as a human normally spends majority of his time in home. Therefore indoor air quality is fundamental for a good health. Most of the infectious diseases are caused due to poor air quality in the inside of buildings for example, school, office, home etc.

1.3 PHOTOCATALYSIS AND NASA

The World Health Organization estimates that urban outdoor air pollution causes 1.3 million deaths worldwide per year, while in developing countries, indoor air pollution causes an estimated 2 million premature deaths. NASA has acknowledged the numerous benefits of photocatalysis. According to an Article “ NASA has studied the benefits of photocatalysis for purifying water during space missions, and plant growth chambers featuring photocatalytic scrubbers have flown on multiple NASA missions, using the photocatalytic process to preserve the space-grown crops by eliminating the rot-inducing chemical ethylene. (The scrubber technology resulted in a unique air purifier, featured in Spinoff 2009, now preserving produce and sanitizing operating rooms on Earth.)”. NASA is of firm belief that photocatalytic materials have the capability to solve issues related to air pollution in the United States.

1.4 TYPES OF PHOTOCATALYSIS

The word “Photocatalysis” is acquired from the Greek Language. Photocatalysis is a gifted method which involves the acceleration of a photoreaction in the presence of light. There are two types of photocatalysis which are homogeneous and heterogeneous.

In Homogeneous photocatalysis, the reactants and photocatalyst exist in the same phase. Transition metal complexes like iron, copper, chromium etc are general examples of homogeneous photocatalysts.

In Heterogeneous photocatalysis, the reactants and photocatalyst exist in different phase from the reactants. The heterogeneous photocatalysis will be discussed in more detail later.

1.5 HETEROGENEOUS PHOTOCATALYSIS

Heterogeneous photocatalysis is based on the irradiation of a semiconductor photocatalyst in contact with a liquid or a gaseous environment. TiO2, ZnO, and CdS are widely used examples (Cassar,2004).

Figure 1 Photocatalysis Mechanism (Saravanan et.al 2017)
A band gap exists between the valence band and conduction band. The electron gets promoted form a valence band to the conduction band in the presence of Ultra violet radiation or fluorescent light. During promotion of electron from a valence band to conduction band a hole is left on the valence band. The holes oxidize the donor molecules. The energy of band gap is 3.2 ev.

\[ D + h^+ \rightarrow D^+ \]

Whereas the conduction band electrons can reduce appropriate electron acceptor molecules.

\[ A + e^- \rightarrow A^- \]

The afore-mentioned explanation presents a brief description to the chemistry of the photochemical reaction in which TiO₂ is seen as the photocatalyst.

2 METAL OXIDES AS PHOTOCATALYST

A photocatalyst is the impetus for the entire photocatalytic process. It is basically a substance that engenders a catalyst activity while utilizing energy from light. There are several examples of metal oxides being used as a photocatalyst such as oxides of vanadium, chromium, zinc, tin, titanium and cerium. Metal oxides are used for photocatalysis and specifically heterogeneous photocatalysis because of their potential to develop charge carriers when they acquire required amount of energy.

2.1 TiO₂ AS A PHOTOCATALYST

TiO₂ is used as a photocatalyst. Although, there are several other metal oxides of vanadium, chromium, zinc, tin and cerium. TiO₂ has ample amount of applications such as it is most widely used inorganic pigment for varnishes and plastics. It is used as a white pigment in paints because of its strong resistance to discoloration under UV light. It is also used in foods, pharmaceuticals and cosmetics. The reason of using TiO2 is that it is very cost efficient and has the capability to go under quick reactions at ambient operating conditions (room temperature, atmospheric pressure). TiO₂ is also used as a photocatalyst because of being chemically stable and compatible with traditional construction materials such as cement. TiO₂ is very effective under weak solar irradiation in various conditions of the environment. The reasons for using TiO₂ as a photocatalyst are that in addition to the self-cleaning characteristics that it shows in the presence of light. It also introduces to its various other characteristics which are the anti-fogging effect, water treatment, air cleaning effect and anti-bacterial effect. Due to these reasons TiO₂ is considered as a very beneficial oxide in the industry. TiO₂ ceramic tiles are considered to be very effective against organic and inorganic materials and also towards bacteria. Hence when such applications of TiO₂ are seen then there comes no question in using any other metal oxide for photocatalytic process.

3 PHOTOCATALYTIC CONCRETE

Photocatalytic concrete is normally composed of cement, water, coarse aggregate, fine aggregate and titanium dioxide either rutile or anatase). TiO₂ occurs naturally as rutile, brookite and anatase. Now the question comes that which one is the best for photocatalysis. Compared with rutile and brookite, anatase shows the highest photoactivity (Benedix et al., 2000). The photocatalytic concept has been induced in the concrete technology which has proven to be a sustainable technique as it maintains the aesthetic characteristics of the structure for a long period of time. There are different examples of structures where photocatalytic concrete has been involved. Such as the New Jubilee Church in Rome. The project was constructed with TX Millennium, a white portland cement with a photocatalytic additive that is manufactured by Italcementi Group. Crushed white marble aggregate was also used to make the concrete a brilliant white. As the project’s “technical sponsor,” Italcementi Group estimates 12,000 man-hours went into developing and testing the new cement to make sure the photocatalytic material is compatible with the concrete and would, indeed, keep the building clean for the thousand-year service life used as the project’s design criteria (Chusid, 2005). For the manufacturing of photocatalytic concrete Photocatalytic cement is an ASTM C150 cement which consist of the photocatalyst in a homogeneous manner. However, there also has been implementation of photocatalytic concrete in the US such as the Bell tower in Dayton, Georgia expouts photocatalytic concrete.
PHOTOCATALYSIS REDUCING POLLUTANTS

The photocatalysis process has always been known for its depolluting effect. Pollution is a serious concern which continuously harming our environment. The major causes of pollution are by car emissions, industries, fossil fuel emissions, and pesticides. There are many pollutants existing in our environment. However the major ones which
are a major threat to our environment include NOx and VOC’s. The above graph shows the benefit of the photocatalysis process as it reduces the overall quantity of NO concentration. NOx is basically the sum of Nitric Oxide (NO) and Nitrogen dioxide (NO2). NOx can cause serious health issues such as in the respiratory system it can cause inflammation of the airways at a high level. NOx also contributes to the formation of fine particles (PM) and ground level ozone, both of which are associated with adverse health effects (Icopal-noxite, 2018). NOx also has a negative effect upon the vegetation. The major source of NOx is from road transport.

5- PHOTOCATALYSIS APPLICATIONS IN HIGHWAY ENGINEERING

According to the EPA, motor vehicles collectively cause 75 percent of carbon monoxide pollution in the U.S. This shows that how massively are roads becoming a source of pollution. In order to mitigate this we need to use photocatalysis for purifying the air. There have been several tests conducted to test the photocatalytic efficiency on the road environment. Such as there is a test conducted in the Netherlands in which one full width of street over a length of 150 m is paved with a concrete pavement having TiO2 in it whereas another part of the street of 100 m was paved with normal paving blocks. The results clearly show the abatement of NOx more than that of the control street. The NOx concentration was, on average, 19% (considering the whole day) and 28% (considering only afternoons) lower than the obtained values in the Control street. Under ideal weather conditions (high radiation and low relative humidity) a NOx concentration decrease of 45% could be observed (Ballari and Brouwers, 2013).

6- CONCLUSIONS

The paper elaborates the different research work carried out by different authors on photocatalysis. All the different practical examples along with the evidence based results indicate towards the advantageous behavior of the photocatalysis process towards the environment. These sustainable applications of photocatalysis whether in the building or road environment truly make it a gifted method along with its cost effectiveness and numerous benefits. The paper also reviews the practical research carried out in the road environment which gives a lucid understanding of the potential in this process. The paper also provides an insight into the threatening situation of increasing pollution levels across the globe. I would like to recommend that as the photocatalytic applications in road environment are still relatively new so more and more research must be conducted to bring the people in confidence regarding the applications for protecting the environment. It is the need of the hour to unite across the borders and adopt this method and all sustainable methods in major projects for a better future for the future generations.
REFERENCES


# The Use of Supplementary Cementitious Materials in Reducing Carbon Dioxide Emissions of Concrete Construction Projects

<table>
<thead>
<tr>
<th>AUTHOR (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
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<tbody>
<tr>
<td>Hala ELIA</td>
<td>Research Assistant</td>
<td>University of Arkansas</td>
<td>United States</td>
</tr>
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<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Amin AKHNOUKH</td>
<td>Associate Professor</td>
<td>East Carolina University</td>
<td>United States</td>
</tr>
</tbody>
</table>

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**KEYWORDS:**

Supplementary Cementitious Materials, Fly Ash, Silica Fume, Carbon Footprint, Compressive Strength

**ABSTRACT:**

The cement industry is a significant CO₂ emitter, contributing approximately 5% of all global CO₂ emissions. These emissions primarily result from the fuel combustion and calcination of raw materials. However, it is possible that using fly ash and silica fume—recycled byproducts from other industries—in the concrete mix will reduce the negative impact of cement industry on the environment. In this study, effects of adding fly ash and silica fume on the compressive strength of concrete containing supplementary cement materials (SCM) were experimentally investigated. The use of SCM results in the production of high-performance concrete (HPC) which leads to reduced cross-section of the concrete structure. Hence, it reduces the amount of cement CO₂ emissions associated with cement manufacturing. In addition to its environmental compliance, produced HPC have higher capacity, improved long-term performance, and reduce the demand to heavy construction equipment due to producing smaller sections with lighter weights.
Cement is an important construction material produced all over the world. During the cement manufacturing process, a huge amount of carbon dioxide (CO\textsubscript{2}) is emitted. Each ton of cement produced results in 0.9 tons of CO\textsubscript{2} (Benhelal et. al., 2012). The cement industry is responsible for around 5% of the world’s man-made CO\textsubscript{2} emissions. Carbon dioxide, the most abundant greenhouse gas, has the greatest impact on global warming (Benhelal et. al., 2012). 61% of global CO\textsubscript{2} releasing are produced by industrial activities such as heat generation, electricity, transportation, and agriculture (IEA, 2010). The main source of CO\textsubscript{2} in cement manufacturing is the calcination process and the combustion of fossil fuels, which are responsible for 90% of CO\textsubscript{2} released from cement production processes (Ishak and Hashim, 2015). Because of the increase of industries all over the world, global greenhouse gas emissions are quickly rising, with CO\textsubscript{2} levels projected to reach 14 gigatons by 2030 (Walsh and Thornley, 2012).

Fly ash—residue from unburned coal—is a byproduct from electrical generation plants. It is taken up by the flue gases, moved out of the burning zone, and collected by electrostatic or mechanical separators (Thomas, 2007). Fly ash is finer than Portland cement and has a spherical shape ranging in size from 10-100 micron. Fly ash is categorized by its chemical composition. Class C ash is high calcium fly ash because it contains more than 20% CaO. Class F ash is low calcium because it has less than 10% CaO. Fly ash is a pozzolan, meaning that when it is mixed with lime (calcium hydroxide or Ca(OH)\textsubscript{2}), which is a byproduct produced from mixing cement and water, it will react to form cementitious compounds. As a result, concrete that contains fly ash is stronger and has higher compressive strength. Also, it has small particles that can perfectly fill voids in the concrete.

Silica fume is an extremely fine non-crystalline silica produced as a by-product of silicon or silicon alloy production. A highly reactive pozzolanic material, silica fume has highly elevated amorphous silicon dioxide content. When cement and water compound together, they produce the binder C-S-H (calcium silicate hydrate) and calcium hydroxide as a byproduct. Silica fume then reacts with the calcium hydroxide to produce an additional binder compound (C-S-H). Silica fume consists of spherical particles or microspheres that are around 100 times smaller than the typical grain of cement (King, 2012). Therefore, introducing silica fume into a concrete mix adds millions of tiny particles that fill the spaces between cement grains. This micro-filling or particle-packing process could significantly improve concrete (Holland, 2015).

Partially replacing Portland cement with industrial byproducts such as fly ash and silica fume decreases the amount of cement required per cubic yard of concrete. Because producing cement requires a large amount of energy, using less cement will decrease the amount of CO\textsubscript{2} emissions and, as a result, the carbon footprint of cement production. Additionally, utilizing fly ash and silica fume in cement will lower the amount of both that must be disposed in landfills. In this study, we added silica fume and fly ash to cement mix as supplementing cementitious materials (SCM) in order to reduce the amount of cement required and, as a result, reduce the amount of CO\textsubscript{2} emitted into the environment. 13 mixes were created and tested for compressive strengthen order to find the ideal amount of fly ash and silica fume to reduce CO\textsubscript{2} emissions while maintaining adequate concrete strength.
2 EXPERIMENTAL INVESTIGATION

Materials
Cement: In this research, Type I Portland cement was obtained company and used in all mixtures to avoid variability in results. The properties of the Portland cement are shown in the Table 1 as obtained from the company.

<table>
<thead>
<tr>
<th>Component</th>
<th>Percent by weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO2</td>
<td>20.08%</td>
</tr>
<tr>
<td>Al2O3</td>
<td>4.65%</td>
</tr>
<tr>
<td>Fe2O3</td>
<td>4.11%</td>
</tr>
<tr>
<td>CaO</td>
<td>63.63%</td>
</tr>
<tr>
<td>MgO</td>
<td>0.94%</td>
</tr>
<tr>
<td>SO3</td>
<td>3.19%</td>
</tr>
<tr>
<td>Na2O</td>
<td>0.16%</td>
</tr>
<tr>
<td>K2O</td>
<td>0.54%</td>
</tr>
<tr>
<td>Limestone</td>
<td>2.7%</td>
</tr>
</tbody>
</table>

Coarse aggregate: The coarse aggregate conforms with the grading requirements of ASTM C-136. It has an absorption capacity of 1.2% and specific gravity of 2.57. Table 2 shows the coarse aggregate gradation.

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>% passing as tested</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5 inch (38 mm)</td>
<td>100</td>
</tr>
<tr>
<td>¼ inch (19.05 mm)</td>
<td>95.1</td>
</tr>
<tr>
<td>3/8 inch (9.5 mm)</td>
<td>28.55</td>
</tr>
<tr>
<td>#4 (4.75 mm)</td>
<td>5.2</td>
</tr>
<tr>
<td>#8 (2.36 mm)</td>
<td>0.4</td>
</tr>
<tr>
<td>#16 (1.18 mm)</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Fine aggregate: Fine aggregate (sand) with a specific gravity of 2.62 and absorption capacity of 0.48% was obtained from Jeffery Sand Co. The fine aggregate complied with ASTM C-33\textsuperscript{15} as presented in Table 3.

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>% Passing as tested</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8&quot; (9.5 mm)</td>
<td>100</td>
</tr>
<tr>
<td>#4 (4.75 mm)</td>
<td>97</td>
</tr>
<tr>
<td>#8 (2.36 mm)</td>
<td>86</td>
</tr>
<tr>
<td>#16 (1.18 mm)</td>
<td>80</td>
</tr>
<tr>
<td>#30 (600 micro meter)</td>
<td>45</td>
</tr>
<tr>
<td>#50 (300 micro meter)</td>
<td>13</td>
</tr>
<tr>
<td>#100 (150 micro meter)</td>
<td>0.5</td>
</tr>
</tbody>
</table>
Fly Ash: Class C fly ash was obtained from Ash Grove Company and had the characteristics represented in Table 4.

Table 4. Chemical compositions of fly ash as obtained from company

<table>
<thead>
<tr>
<th>Chemical Component</th>
<th>Percent by weight %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silica (SiO2)</td>
<td>40</td>
</tr>
<tr>
<td>Alumina (Al2O3)</td>
<td>16</td>
</tr>
<tr>
<td>Ferric Oxide (Fe2O3)</td>
<td>6</td>
</tr>
<tr>
<td>Calcium Oxide (CaO)</td>
<td>24</td>
</tr>
<tr>
<td>Magnesium Oxide (MgO)</td>
<td>2</td>
</tr>
<tr>
<td>Sulfate Oxide (SO3)</td>
<td>3</td>
</tr>
<tr>
<td>Loss of ignition (LOI)</td>
<td>6</td>
</tr>
</tbody>
</table>

Silica Fume: Silica fume was obtained from Elkem Company; Table 5 shows its chemical characteristics.

Table 5. Chemical composition of silica fume as obtained from company

<table>
<thead>
<tr>
<th>Chemical composition</th>
<th>Percent by weight %</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO2</td>
<td>94.3</td>
</tr>
<tr>
<td>Al2O3</td>
<td>0.09</td>
</tr>
<tr>
<td>Fe2O3</td>
<td>0.1</td>
</tr>
<tr>
<td>CaO</td>
<td>0.3</td>
</tr>
<tr>
<td>MgO</td>
<td>0.43</td>
</tr>
<tr>
<td>SO3</td>
<td>---</td>
</tr>
<tr>
<td>K2O</td>
<td>0.83</td>
</tr>
<tr>
<td>Na2O</td>
<td>0.27</td>
</tr>
</tbody>
</table>

High Range Water Reducer (HRWR): HRWR was obtained from Euclid Chemical Company to reduce the amount of water in the concrete mix and obtain high compressive strength by reducing the amount of water/cement ratio.

There are 13 mixes of concrete in this research; one was the control mix without SCM, and the other mixes contained SCM. Table 6 shows all the mixes used in this paper. Fig. 1 represents the compressive strength of the control mix after 1, 3, 7, 14, and 28 days. Fig. 2 shows the compressive strength values for all SCM mixes after 1, 3, 7, 14, and 28 days.
### Table 6. Mixes of concrete

<table>
<thead>
<tr>
<th>Material</th>
<th>control</th>
<th>Mix 1</th>
<th>Mix 2</th>
<th>Mix 3</th>
<th>Mix 4</th>
<th>Mix 5</th>
<th>Mix 6</th>
<th>Mix 7</th>
<th>Mix 8</th>
<th>Mix 9</th>
<th>Mix 10</th>
<th>Mix 11</th>
<th>Mix 12</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>730</td>
<td>1050</td>
<td>1050</td>
<td>1050</td>
<td>1050</td>
<td>1050</td>
<td>1050</td>
<td>1050</td>
<td>1050</td>
<td>1050</td>
<td>1125</td>
<td>1050</td>
<td></td>
</tr>
<tr>
<td>Cement</td>
<td>1994</td>
<td>2440</td>
<td>2440</td>
<td>1700</td>
<td>2440</td>
<td>1460</td>
<td>1740</td>
<td>2250</td>
<td>2450</td>
<td>1600</td>
<td>2250</td>
<td>1600</td>
<td></td>
</tr>
<tr>
<td>Fine aggregate</td>
<td>983</td>
<td></td>
<td></td>
<td>725</td>
<td></td>
<td>625</td>
<td></td>
<td>740</td>
<td></td>
<td>675</td>
<td></td>
<td>675</td>
<td></td>
</tr>
<tr>
<td>Coarse aggregate</td>
<td></td>
<td>150</td>
<td>150</td>
<td>150</td>
<td>150</td>
<td>150</td>
<td>150</td>
<td>150</td>
<td>150</td>
<td>150</td>
<td>240</td>
<td>150</td>
<td></td>
</tr>
<tr>
<td>Fly ash</td>
<td>150</td>
<td>40</td>
<td>25</td>
<td>25</td>
<td>25</td>
<td>40</td>
<td>60</td>
<td>35</td>
<td>60</td>
<td>70</td>
<td>70</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silica fume</td>
<td>400</td>
<td>260</td>
<td>230</td>
<td>285</td>
<td>285</td>
<td>280</td>
<td>230</td>
<td>225</td>
<td>260</td>
<td>240</td>
<td>240</td>
<td>235</td>
<td></td>
</tr>
<tr>
<td>HRWR</td>
<td>1020</td>
<td>9172</td>
<td>8390</td>
<td>8175</td>
<td>8310</td>
<td>8820</td>
<td>8010</td>
<td>8255</td>
<td>9330</td>
<td>8978</td>
<td>9870</td>
<td>9020</td>
<td>9772</td>
</tr>
<tr>
<td>Water</td>
<td>4720</td>
<td>1478</td>
<td>1266</td>
<td>1388</td>
<td>1324</td>
<td>1195</td>
<td>1270</td>
<td>1590</td>
<td>1410</td>
<td>1710</td>
<td>1440</td>
<td>1610</td>
<td></td>
</tr>
<tr>
<td>24hrs comp. st. (psi)</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>28days comp. st. (psi)</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>

3. TESTING OF SPECIMENS

Compressive strength was used to study the ability of fly ash and silica fume to reduce the amount of cement needed to produce strong concrete. Compressive strength was measured according to ASTM C39/C39M-17b. A 28-day compressive strength study was carried out with all the SCM-cement mixes, as well as the control mix that did not contain SCMs. Fly ash and silica fume significantly affected the compressive strength of the concrete. For the control mix, the compressive strength value was 4720 psi. For the mixes containing fly ash and silica fume, the compressive strengths ranged from 10550 to 17100 psi. The compressive strength represents an inverse relationship with a cross-section area of the concrete structure, which means that when the compressive strength is high, the cross-section area will be smaller, leading to a reduced number of concrete materials required, especially cement. Figures 1 and 2 shows the compressive strength testing results for different concrete mixes.

![Figure 1. Compressive strength vs. time for control mix](image)
4. RESULTS AND DISCUSSIONS

The results of this study enabled us to calculate the amount of cement needed in each supplemented mix and to then compare this with the amount of cement in the control mix. For example, by dividing the compressive strength of Mix1 by the compressive strength of the control mix \[14780/4720 = 3.13\], we found that the SCM mix has greater strength than the traditional mix. To achieve the same compressive strength as the control mix, the amount of cement would have to be multiplied by \[3.13\times 3.13 = 2284\] lbs. of cement, in comparison with the 1050 lbs. used in the SCM mix. Therefore, using Mix1 uses around 54% less cement, as shown in Fig. 3.

Based on these results, we can estimate the real reduction of cement production in the 5 countries that make the most cement. Producing one ton of cement causes 0.9 tons of CO$_2$ emissions. Therefore, if cement production is reduced by 54%, the CO$_2$ emission will be reduced by about 54% as well. Fig.3 shows the projected cement reduction annually by country if SCM is added in the mix. If Mix 1 is used, cement production in China would decrease from 2410 to 1301 million tons and, in the USA, from 86 to 46 million tons—representing about 46% cement reduction. Mix1 could reduce cement production from 2410 to 1108 million tons for China and, for the USA, from 86 to 40 million tons. As a result, in China, CO$_2$ emissions would be reduced from 2169 to 997 million tons (Fig. 4) and, in the USA, from 77 to 36 million tons.

The same type of data and projections were recorded for each Mix (Fig. 3 and 4). By using Mix 3, cement production would be reduced from 86 to 51 million tons in the USA, a 41% decrease. Mix 4 would reduce the amount by 45%, from 86 to 47 million tons in the USA and from 77 to 42 million tons in Turkey. Mix 5 performed even better, having the potential to reduce cement production by 49%. Mix 6 would reduce cement by 36% in the USA, from 86 to 55 million tons. Mix 7 could reduce cement by 42%—from 86 to 50 million tons in the USA and from 60 to 35 million tons in Brazil. Mix 8 could reduce the amount of cement from 86 to 37 million tons in the USA (a 57% decrease) and from 290 to 125 million tons in India. By using Mix 9, cement would be reduced from 86 to 42 million tons in the USA (52% decrease). Cement production would be reduced by 60% by Mix 10, making it the optimum mix in this research; this mix would reduce cement production from 86 to 34 in the USA and from 2410 to 964 million tons in China. Mix 11 and Mix 12 would reduce cement production by 53% and 58%, respectively.
Fig. 4 represents the projected concrete production-related CO₂ emission reduction by country if the tested SCM mixes of concrete replaced current concrete production. Mix 1 could reduce CO₂ from 2169 to 1171 million tons in China and from 77 to 41 million tons in the USA. Mix 3 could reduce CO₂ emission in the USA from 77 to 46 million tons and, in India, from 261 to 154 million tons. Mix 4 would reduce CO₂ emissions by 45%, lowering the amount in the USA from 77 to 42 million tons and, in Turkey, from 69 to 38 million tons. Mix 5 can reduce the amount of CO₂ from 77 to 50 million tons in the USA, while Mix 6 can reduce it from 2169 to 1388 million tons in China. CO₂ emission can be reduced from 77 to 45 million tons in USA by using Mix 7. Mix 8 can reduce USA CO₂ emissions from 77 to 33 million tons and India emissions from 261 to 112 million tons. CO₂ emission can be reduced from 77 to 38 for the USA by using Mix 9. Mix 10 would reduce CO₂ emission from 2169 to 868 million tons in China and from 77 to 31 million tons in the USA. CO₂ emission in China can be reduced from 2169 to 1020 million tons and from 77 to 36 million tons in the USA by Mix 11. Finally, Mix 12 would reduce CO₂ emission from 77 to 32 million tons in the USA and from 2169 to 911 million tons in China. Therefore, the optimum Mix is Mix 10, which reduced the amount of cement by 60%, which would result in a 60% reduction in CO₂ emission into the environment.
5. CONCLUSION

Using supplementary cementing materials allows the amount of cement in a concrete mix to be greatly reduced, up to 36 to 60%. The optimum mix of concrete shows 60% reduction of the amount of cement in the concrete mix, which would also reduce CO$_2$ emissions from cement manufacturing by 60%. Therefore, by integrating fly ash and silica fume into concrete production, we can help the environment by not only recycling these materials from other manufacturing processes, but also by reducing one of the major global contributors of greenhouse gases.

REFERENCES

Walsh, C. and Thornley, P. (2012). Barriers to improving energy efficiency within the process industries with a focus on low grade heat utilization. J. Clean. Prod., 23(1), PP.138-146
**PAPER TITLE**
Maintenance of Porous Friction Course Pavements

**TRACK**

<table>
<thead>
<tr>
<th>AUTHOR (Capitalize Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
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<tr>
<td>Yetkin YILDIRIM</td>
<td>CTO</td>
<td>Terra Pave International</td>
<td>USA</td>
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<tr>
<td>Gokhan SAYGILI</td>
<td>Assistant Professor</td>
<td>The University of Texas at Tyler</td>
<td>USA</td>
</tr>
</tbody>
</table>

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**KEYWORDS:**
Maintenance, porous friction course, permeability, penetration, tensile strength, pull-off, polymer, asphalt emulsion

**ABSTRACT:**
A porous friction course (PFC) is a thin layer of permeable asphalt mixture with advantages of increased permeability, reduced road noise, and improved ride qualities. To represent the maintenance of PFC pavements, specimens were treated with additives including a polymer (TF), asphalt emulsions, SS-1 (ionic) and CSS-1 (cationic), and polymer-modified asphalt emulsion (CSS-1P). Laboratory testing was performed on the permeability, penetration, tensile strength, and pull-off characteristics of PFC specimens. The results revealed that TF significantly increased the penetration, tensile strength, and pull-off strength. The tensile strengths of CSS-1 and CSS-1P were comparable. The closest permeability condition with respect to the control test was achieved on TF specimens. However, the permeability of CSS-1P was around 30% greater than the permeability of the specimens with CSS-1. Penetration tests revealed that SS-1 and CSS-1 did not penetrate into the base material but formed a sticky coat on the surface of the reference sand.
Maintenance of porous friction course pavements

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1 INTRODUCTION

Porous friction course (PFC) roads are revolutionary freeways that can withstand all types of climate conditions, whether it be flooding or extreme heat. Not only do these types of roads help ensure safety to their travelers and reduce noise, but they also benefit the environment by absorbing the rainfall and naturally filtering and guiding it down streams, ponds, and lakes. PFC road surfaces are safer to drive on during wet weather conditions because of their ability to absorb rainfall upon contact. This action prevents the drastic reduction of the force of friction, which makes it harder for cars to slide off track, thus decreasing wet pavement accidents. PFC road surfaces also reduce road noises caused by tires rolling on the street. When a tire travels on ordinary pavement, air is forced towards the front and back, thereby, spreading noise into its surroundings. However, the empty spacing in PFC road surfaces allows the air to be pumped downward, and absorbed into the road.

There has been significant research on the feasibility of porous asphalt pavements. Constructed under an Environmental Protection Agency (EPA) grant, the Woodlands, Texas was one of the world’s first porous asphalt pavement field implementations. (HUD 1972). Studying the behavior of pervious concrete pavement, Henderson and Tighe (2010) concluded that it is possible to achieve a high quality pervious concrete pavement in freeze-thaw climates. The experimental testing program carried out by Jiang et al. (2015) on porous asphalt concrete revealed that air voids significantly affect the performance of porous asphalt concrete. In an effort to increase the strength properties of Portland cement pervious concrete, Huang et al. (2010) suggested to include a combination of latex polymer and granular soil (e.g. sand) into the mixture.

Texas Department of Transportation (TxDOT) defined a new-generation gap-graded friction course, entitled Porous Friction Course (PFC), with high percentage of air voids (TxDOT Item 342). Placed at the surface of a pavement structure in a thin layer, PFC’s provide increased permeability, reduced pavement noise, and improved traffic safety due to the reduced hydroplaning, splash and spray, and glare under wet conditions. A performance evaluation of the porous asphalt field test sections constructed on the Interstate Highway 74, east of Indianapolis, revealed that a porous friction course produces significantly lower noise levels as compared to conventional hot mix asphalt surface (McDaniel 2004). The service life of a PFC can range from 7 to 10 years (Estakhri et al. 2008). The disadvantages of PFC roads include high maintenance costs and less durability which can be defined as resistance to raveling and possibly rutting and cracking (Liu et al. 2010).

In this research, to represent the maintenance of PFC pavements, a laboratory testing program was carried out on PFC specimens treated with additives including TF, SS-1, CSS-1, and CSS-1P. A research effort was made to optimize the permeability and strength so that PFC specimens are not only permeable but also strong enough to support traffic loading. Permeability, penetration, tensile strength, and pull-off tests were performed to evaluate the mechanical properties of treated PFC specimens.

2 MATERIALS

In this study, water-based Terra Fog liquid polymer (TF) is used. TF is in the form of aqueous dispersion. The pH value is around 3.5 - 6.5 and the density is around 0.99 g/cm\(^3\) (68 °F (20 °C)) (data for Water (7732-18-5)). Specific Gravity (Relative density) is around 0.97 - 1.10 Water=1 (liquid). TF is provided by Terra Pave International which is located at the University of Texas at Austin. Two types of asphalt emulsions; SS-1 (ionic) and CSS-1 (cationic) and polymer-modified asphalt emulsion (CSS-1P) were used in this work. These are commonly used slow set asphalt emulsions designed for use as a tack coat and base stabilization.
Two types of aggregates were used throughout the laboratory testing program. These aggregates were trap rock #5 and crushed limestone screening dust. Trap rock is a fine-grained, non-granitic intrusive or extrusive igneous rock. It is typically used as a crushed rock in road construction. Crushed limestone screening dust is the finest form (i.e. powder). It serves as a binding agent in asphalt. These aggregates were homogenously mixed using a mix proportion of crushed limestone screening dust: trap rock #5 = 1: 6.5, by weight.

3 TEST PROCEDURES

Permeability, tensile strength, and pull-off tests were performed on specimens compacted at the optimum moisture content. To obtain the compaction curves, aggregates were first homogenously mixed and compacted in a standard mold having a capacity of 1/30 ft³, an internal diameter of 4 inches, and a height of 4.584 inches. Aggregates in the mold were compacted in three layers with 25 blows per layer from a 5.5 pound hammer dropped from a height of 12 inches. Based on the compaction tests performed at various moisture content values, the optimum moisture content and the maximum dry density were determined to be around 11% and 121 pcf, respectively.

3.1 MIXTURE PRODUCTION

The surface of compacted specimens were covered by TF, SS-1, CSS-1, and CSS-1P. As reported in Freeman et al. (2010), typical application rates for emulsions vary between 0.2 to 0.5 gal/yd² (0.9 to 2.3 l/m²). As shown in Figure 1, additives were covered evenly on the top surface of the compacted specimens. The process was slow enough so that compacted specimens absorbed the additives uniformly and left behind a thin and quick drying film on the surface without any puddles.

3.2 TENSILE STRENGTH TEST

Indirect tensile strength tests were performed on specimens compacted at the optimum moisture content. Following TxDOT specifications (TxDOT Tex-226-f, 2005), the loading rate for the test was selected as 2 in/min. Indirect tensile strength tests were performed on extruded specimens.

3.3 PULL-OFF TESTS

Following TxDOT surface treatment bond test procedure, pull-off tests were executed on flat and clean specimen surfaces to improve the accuracy of tests. The interface was first bonded with an epoxy and allowed to fully cure. Next, the dolly was pulled off perpendicularly to the test surface. The force required to pull the coating off the surface was measured.
3.4 PERMEABILITY TESTS

To measure the permeability of the specimens, an approach similar to the ASTM D5856 test method for permeability was used. In this approach, 500 ml of water was poured onto the surface of each specimen and allowed to stand for complete drainage. Assuming a constant hydraulic gradient (i.e. steady-state conditions), the coefficient of permeability was calculated using Darcy’s equation.

3.5 PENETRATION TESTS

Sand penetration tests were performed to determine how much polymer and asphalt emulsion would penetrate into the reference sand placed as the base course. Five grams of the polymer and asphalt emulsions were poured onto the sand surface at a constant speed from a height of approximately 11 cm. The specimens were allowed to stand for 24 hours. Then specimens were cut in the vertical direction and the penetration depth into the depth was measured using a Vernier caliper.

4 RESULTS

4.1 TENSILE STRENGTH TEST

Three PFC specimens were prepared with TF, CSS-1, and CSS-1P to evaluate the tensile strength. Figure 2 displays the tensile strength test results. The results showed that the tensile strength of TF was around 2.5 times the tensile strength of CSS-1 and CSS-1P. The tensile strength of CSS-1 and CSS-1P were comparable meaning that polymer-modification did not change the tensile strength characteristics of the specimen with asphalt emulsion.

![Figure 2. Tensile strength test results](image)

4.2 PULL-OFF TESTS

To evaluate the pull-off strength, three PFC specimens were prepared with TF, SS-1, and CSS-1. The results displayed in Figure 3 revealed that the pull-off capacity of TF was more than 4.5 times that of SS-1 and CSS-1. The pull-off strength of CSS-1 was slightly greater than that SS-1; However, the pull-off capacity of the specimens with asphalt emulsions (i.e. SS-1 and CSS-1) were significantly lower when compared to that of the TF.
4.3 PERMEABILITY TESTS

Three PFC specimens were prepared with TF, CSS-1, and CSS-1P to evaluate the permeability. The results showed that the permeability of the TF was around 2.1 times the permeability of CSS-1 and 1.6 times the permeability of CSS-1P. The permeability of CSS-1 was around 30% greater than the permeability CSS-1P. The closest permeability condition with respect to the control test was achieved on TF specimens. Table 1 displays the corresponding coefficient of permeability values.

Table 1. Permeability test results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Coefficient of permeability (ft/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TF</td>
<td>0.0073</td>
</tr>
<tr>
<td>CSS-1</td>
<td>0.0045</td>
</tr>
<tr>
<td>CSS-1P</td>
<td>0.0034</td>
</tr>
<tr>
<td>Reference (no additive)</td>
<td>0.0146</td>
</tr>
</tbody>
</table>

5. DISCUSSION

The laboratory test results clearly show that TF is a better PFC additive as it comparatively provides the greatest strengths (tensile and pull-off) and penetration as well as the closest permeability condition with respect to the control test. Compared to SS-1, CSS-1, and CSS-1P; TF is less thick allowing the specimen to be able to drain the liquid faster and its runny nature allows for the liquid to spread through the aggregates and bind them together more with greater strength.

Regarding the environment, the asphalt emulsions contain volatile organic compounds (VOCs). This material is known to have many short term symptoms but can also lead to significant health problems. TF, however, does not contain VOCs. Roughly, the application of asphalt emulsion on the surface of roads releases 60 kg/ton of greenhouse gases whereas TF only releases 3 kg/ton.

6. CONCLUSION

Laboratory testing was carried out to investigate the permeability, penetration, tensile strength, and pull-off characteristics of PFC specimens treated (maintained) using TF, SS-1, CSS-1, and CSS-1P. The following observations were made:

- The tensile and pull-off strengths increased after the application of the additives but the TF increase was stood out significantly above the others. The tensile strengths of CSS-1 and CSS-1P were comparable. The polymer modification on the asphalt emulsion seemed to have only a minor effect on
The tensile strength. The pull-off capacity of the specimens with asphalt emulsions (i.e. SS-1 and CSS-1) were significantly lower when compared to that of TF.

- The closest permeability condition with respect to the control test was achieved on TF specimens. The permeability of CSS-1 was approximately 30% greater than the permeability CSS-1P.
- The penetration of TF was significantly greater than that of the SS-1 and CSS-1. Asphalt emulsions did not penetrate into the base material but formed a sticky coat on the surface of the reference sand.

7 REFERENCES


TxDOT Designation: TEX-226-F. Indirect tensile strength test. Construction Division, Texas Department of Transportation; 2010.

Development of environment friendly cold mix asphalt repair material

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ABSTRACT

Most of the roads in Japan were built during the period of high economic growth in the 1960s. Now about 50 years old, the stock of roads is in need of renewal all at the same time. Minor repair of pavement is performed using bagged cold asphalt repair materials, but they are not as strong as the more common hot asphalt mixtures and are prone to cause problems such as rutting and aggregate scattering at an early stage. To overcome these challenges, the authors developed bagged cold asphalt mixture with a level of strength comparable to that of hot asphalt mixtures. Because this cold asphalt mixture develops its strength upon being sprayed with water, it may be used where paving work is performed in the rain or on a road surface with puddles of water. This technology has also found its way into growing applications, including cold asphalt mixtures bagged in flexible containers for the pavements on a remote island, cold slurry asphalt for the repair of poured asphalt on steel plate decks, and the repair of running lamp pipes on airport runways. These products can all be used in ambient temperatures, which allow for much lower temperatures during paving work compared to hot asphalt mixtures. Cold asphalt mixtures are repair materials that help improve working environment at the paving site.

1. INTRODUCTION

Bagged cold asphalt repair materials (hereafter the "cold asphalt mixtures") are used for different purposes such as repair of potholes and bumps in pavements making paving work easy and storage stability high. However, because their strength development is slow compared to hot asphalt mixtures and durability is also poorer, cold asphalt mixtures have been mainly used for the temporary rehabilitation of lightly trafficked roads. Meanwhile, the recent years have seen efforts being made to improve the durability of cold asphalt mixtures in the aim to apply them for the repair of roads with relatively heavy traffic, as well as for mid- and long-term temporary and permanent rehabilitation works. This has led to the development of various highly durable products. Unfortunately, the strength, development time, and durability of cold asphalt mixtures have yet to reach the level of hot asphalt mixtures, and frequently cause damages such as rutting and aggregate scattering immediately after paving work and during summer. There are also limits to the use of cold asphalt mixtures in paving work during rain or when the repaired area is contaminated with puddles of water.

To overcome these challenges, the authors developed highly durable all-weather cold asphalt mixture that has a level of strength comparable to that of hot asphalt mixtures and may be used where paving work is performed in the rain or on a surface with puddles of water. It has since been successfully put into commercial applications. This article reports a summary of the properties, scope of applications, examples of its uses in paving work, and application technologies of this newly developed cold asphalt mixture (hereafter the "new mixture").

2. CONVENTIONAL COLD ASPHALT MIXTURES

Conventional cold asphalt mixtures rely on cutback asphalt using a petroleum solvent to ensure its workability at ambient temperatures. These mixtures develop their strength when the solvent volatilizes after the paving work has completed. However, the volatilization of petroleum solvent takes several days to several months. Since the strength development is slow, some repaired areas may be damaged soon after the paving work, or even several months after the repair, particularly during summer, because the lubricating oil inside the pavement remains without volatilizing. In addition, when paving work is carried out in the rain or in water puddles, the water causes the durability of the mixture to be reduced as well as the lubricating oil to leach out, resulting in many cases of paving problems, and creating environmental issues.
3. OUTLINE OF THE NEW MIXTURE

3.1 Mechanism of Strength Development

Figure 1 shows the mechanism of strength development of the cold asphalt mixture developed. By coating the asphalt surface with special non-petroleum lubricating oil, workability in normal temperature can be ensured. In addition, by spraying water after laying down asphalt (before rolling), the special lubricating oil, reaction assisting material, and water chemically react and harden, and the mixture undergoes strength development. As the special lubricating oil and asphalt have very good compatibility. The special lubricating oil and products made of reaction assisting material have the same or more strength than asphalt at service temperatures, the mixture can acquire high durability equivalent to or above hot asphalt mixtures after the paving work. Furthermore, as the speed of the chemical reaction is faster than the volatilization of petroleum solvent used for cold asphalt mixtures, the strength development of the mixture is also fast, promising early traffic release after the paving work.

3.2 Manufacturing Method of the New Mixture

The new mixture can be manufactured using the same method as normal hot asphalt mixtures. However, it should be noted that mixing temperature during the manufacture must be above 110°C because the chemical reaction between the lubricating oil and reaction assisting material initiates with the slightest amount of water, which may cause the mixture to harden due to the residual moisture in the aggregate.

3.3 Mixture Types

In fact, various particle sizes are used. Although the dense grade with maximum aggregate particle size of 13 mm is the major type. The product series provides a range of particle sizes to meet different needs. However, the porous type has lower strength compared to the continuous grading type because the coarse aggregate serves as spot glue, and is thus difficult to use for places where rest steering and torsional effects of tires are frequent. We are therefore currently investigating the application of the mixture as repair material for porous asphalt pavements for roadways.
4. INDOOR EVALUATION TEST

4.1 Evaluation procedure
There is no prescribed method for evaluating the properties about the strength and durability of bagged cold asphalt mixtures. For this reason, the properties of the new mixture were evaluated by the Marshall Stability test and the wheel tracking (WT) test conducted on hot asphalt mixtures. By conducting these tests, it enables to numerically evaluate the strength and durability of the new mixture, and to compare with the same index as the hot asphalt mixture.

(1) Marshall Stability test

The Marshall Stability test evaluates resistance (stability) to deformation of asphalt mixture due to the load of transportation vehicles.
A cylindrical test piece (Photo 1) with a diameter of about 10.2 cm and a height of about 6.3 cm was used. Load the test piece in a compacted state and determine the maximum load (Marshall Stability) indicated until the test piece got broken. The test situation is shown in Photo 2. The test result to be reported from now uses the average value of three evaluated mixtures.

(2) Wheel tracking test

The wheel tracking test is performed mainly to examine the Dynamic Stability: DS (pass / mm) which is an index of the flow resistance of the asphalt mixture.
The shape of the test piece is length × width × thickness = 30 × 30 × 5 cm (Photo 3). By repeatedly running wheels with a ground pressure of 680 N on the test piece, a test on the flow resistance (rutting resistance) of the asphalt mixture is carried out. The test situation is shown in Photo 4. The results reported here also use the average of three evaluated mixtures.
4.2 Properties of Mixture

The test temperatures were 20°C and 60°C.

(1) Comparison with conventional cold asphalt mixtures

To evaluate the mixture strength immediately after paving and after placing the road in service, Marshall Stability test was conducted at 20°C one hour after preparing the test samples and seven days after curing at 60°C. WT test were also carried out for seven days after 60°C-curing. General type and highly durable type were used as the cold asphalt mixtures for comparison.

Figure 2 and figure 3 show the results. The new mixture was found to have very high strength compared to the other two cold asphalt mixtures for both Marshall Stability and dynamic stability one hour after preparing the test samples and seven days after curing at 60°C.

(2) Comparison with hot asphalt mixture

To compare properties of the new mixture with those of hot asphalt mixtures, Marshall and WT test were conducted at 60°C seven days after 20°C-curing. Taking the particle size of the hot asphalt mixture to be the same as that of the new mixture, straight asphalt 60/80 binder was used.

Table 1 shows the results. The new mixture indicated values equivalent or above the hot asphalt mixture, confirming that the new mixture has the same strength as hot asphalt mixtures even though it is a bagged cold asphalt mixture.

Table 1. Properties of mixture

<table>
<thead>
<tr>
<th>Item</th>
<th>New mixture</th>
<th>Hot asphalt mixture</th>
<th>Specification*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Marshall Stability at 60°C (kN)</td>
<td>9.2</td>
<td>9.8</td>
<td>≥4.9kN</td>
</tr>
<tr>
<td>Dynamic Stability at 60°C (pass/mm)</td>
<td>6,000</td>
<td>500</td>
<td>-</td>
</tr>
</tbody>
</table>

※ : Guideline for pavement design and construction

4.3 The Speed of Strength Development

The new mixture develops strength not by the volatilization of the petroleum solvent but by chemical reaction. As its chemical reaction time is faster than volatilization time, strength development of the new mixture is faster than that of normal cold asphalt mixtures, enabling quick traffic release.
(1) **Initial strength of mixtures**

The speed of strength development of the new mixture was verified by Marshall Stability test at 20°C. Figure 4 shows the test result. The strength of the mixture increases with time, confirming that the Marshall stability of the mixture was higher than the general type (20°C, 7-day curing) 30 minutes after sample preparation, and higher than highly durable products (20°C, 7-day curing) one hour after.

(2) **Durability immediately after paving work**

To verify the strength development of the new mixture immediately after paving work, a car was driven over the road immediately after pothole repair, as shown in photo 5, and the road surface profile before and after the tires had run over the road were measured, as shown in photo 6. Table 2 and table 3 include the test conditions and results. Rutting is less compared to conventional cold asphalt mixtures, indicating that the mixture has superior initial durability. In addition, there were no cracks caused by aggregate scattering or displacement.

![Figure 4. Result of verifying strength development](image)

![Photo 5. Road condition](image)

![Photo 6. Measurement road surface profile condition](image)

<table>
<thead>
<tr>
<th>Table 2. Measuring conditions</th>
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<tr>
<td><strong>Item</strong></td>
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<tr>
<td>Paving scale</td>
</tr>
<tr>
<td>Rolling compaction method</td>
</tr>
<tr>
<td>Running vehicle</td>
</tr>
<tr>
<td>Vehicle used</td>
</tr>
<tr>
<td>No. of runs (Running time)</td>
</tr>
<tr>
<td>Road surface profile</td>
</tr>
<tr>
<td>measuring device</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Table 3. Rut depth</th>
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<tr>
<td><strong>Type</strong></td>
</tr>
<tr>
<td>New mixture</td>
</tr>
<tr>
<td>General product</td>
</tr>
<tr>
<td>Highly-durable product</td>
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</tbody>
</table>
4.4 Workability in Low Temperatures

Table 4 shows the results of verifying workability of the new mixture in low temperatures. Evaluation was carried out by Marshall Stability test at 20°C and handling during work. In this study, the mixture temperature was decreased to 0 and -10°C, so the compaction density was slightly lower than when paving was carried out at 20°C. However, there was essentially no drop in workability and mixture’s Marshal Stability, confirming that there is no sharp decrease in durability even in paving at low temperatures. As the freezing of the sprayed water during paving can cause inadequate compaction, there is a need to carry out compaction quickly.

<table>
<thead>
<tr>
<th>Compaction temperature (°C)</th>
<th>Density (g/cm³)</th>
<th>Compaction degree compared with 20°C (%)</th>
<th>Workability (handling)</th>
<th>Marshall stability at 20°C (kN)</th>
</tr>
</thead>
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<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3-hour curing</td>
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<tr>
<td>20</td>
<td>2.342</td>
<td>100</td>
<td>Good</td>
<td>10.5</td>
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<tr>
<td>0</td>
<td>2.314</td>
<td>98.8</td>
<td>Good</td>
<td>9.7</td>
</tr>
<tr>
<td>-10</td>
<td>2.300</td>
<td>98.2</td>
<td>Good</td>
<td>9.5</td>
</tr>
</tbody>
</table>

4.5 Storage stability

As the new mixture develops strength by adding water, it can start reacting gradually even with the slightest amount of water such as moisture in the air, etc. Ideally the mixture should be stored, for example, in airtight bags. Table 5 shows the results of verifying storage stability. When bags used for storing conventional cold asphalt mixtures are sealed by stitching with a sewing machine, hardening started one to two months after storage. However, when storage bags with better humidity prevention and waterproofing performance were used, it was confirmed that sealing the bags by thermal compression to increase airtightness enabled storage for six months. In addition, workability and properties of a mixture which had been stored for six months were checked, and it was found that conditions were good and not different from mixtures just manufactured.

<table>
<thead>
<tr>
<th>Type of bag</th>
<th>Sealing method</th>
<th>Hardening rate of mixture in bag (%)</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>1 month</td>
<td>2 months</td>
<td>3 months</td>
<td>6 months</td>
</tr>
<tr>
<td>Conventional type</td>
<td>Stitching by sewing</td>
<td>0</td>
<td>20</td>
<td>70</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>Dedicated type</td>
<td>Termocompression</td>
<td>0</td>
<td>0</td>
<td>50</td>
<td>80</td>
<td></td>
</tr>
</tbody>
</table>

5. PAVING PROCESS

The paving process with the new mixture is the same as normal cold asphalt mixtures other than the spraying of water before compaction. The following describes the paving process using pothole repair as an example.
① Pouring and Spreading the Mixture

The required amount is poured over the area to be paved. The shrinkage allowance is about 30%. The paved area can be contaminated by puddles of water. Although applying of tack coat is not required, it is still performed as necessary because it improves durability. The mixture is then spread as done with normal cold asphalt mixtures, however, efforts must be made to prevent the coarse aggregate from concentrating in one place as this causes aggregate scattering.

② Water Spraying

Water must always be sprayed before rolling. As shown in photo 7, water is sprayed using a plastic bottle, watering can, etc. over the whole surface evenly. The amount of water to be sprayed is 1 liter per bag of mixture (20 kg), and can be slightly more or less than this. However, more will not cause any problems, but inadequate spraying can delay strength development.

③ Rolling

Rolling was carried out promptly after water spraying. As shown in photo 8, compaction by stepping with the foot is adequate. Durability is better when rolling machines such as plate compactor, etc. are used. If rolling is delayed, chemical reaction with the mixture progresses, resulting in paving problems. For this reason, rolling need to be carried out promptly.

④ Traffic Release

In small-scale paving work such as repair of potholes, etc., traffic release may be possible after completion of the paving work. However, curing may need to be carried out for about one to two hours when large cars are scheduled to frequently run over the areas where torsional effects of tires occur or at large areas of paving work.

6. EXAMPLES OF APPLICATION

The new mixture has been used in repair work in a number of cases with varying degrees of pavement damage, and its good serviceability maintained. The following are a few examples of its applications.

6.1 Restoration Work after Earthquake

Photo 9 shows that the new mixture was used for repairing potholes that had formed on roads in Miyagi Prefecture during the Great East Japan Earthquake. Conditions remain good even after two years from paving.

6.2 Restoration Work after North Kyushu heavy Rain Disaster

As shown in photo 10, a torrential downpour which occurred in the northern part of Kyushu in July 2012 severely damaged roads in Fukuoka Prefecture. Photo 11 shows that the new mixture was used in restoration work to repair the roads. More than 100 bags (containing 20 kg) were used.
6.3 Application to Paved Sites with Torsional Effect of Tires (Exit/entrance of distribution center)

Photo 12 shows the repair situation at the entrance/exit of a distribution center. There is an area showing torsional effect of large vehicles. Repairs had been carried out several times but the area quickly became damaged again after each repair. When it was repaired using the new mixture which has high durability, good serviceability was confirmed.

7. NEW ENDEAVORs

The new mixture can be applied for various purposes using techniques developed for it. Some new methods of use are introduced below.

7.1 Use as a hot asphalt mixture

The new mixture is suitable for use without being bagged for up to twelve hours after its production. This means that it may be used in lieu of a regular hot asphalt mixture on the day it is produced. Since the new mixture is produced at around 110°C, namely the temperatures at which there is no or nearly no residual water in aggregates, it allows for the manufacturing temperatures to be reduced by about 50°C than with regular hot asphalt mixtures. This should not only cut down on CO₂ emission but also help improve the working environment for workers, for instance by protecting them against heat stroke.

Faster chemical reaction between a special lubricating oil and a reaction assisting material means that the finished asphalt develops strength more quickly, which in turn allows early traffic release. In addition, by making the shipment temperature the same as that of regular asphalt mixtures, the new mixture may also be used as an improved-workability mixture for paving work where the temperature of a mixture is expected to drop, for instance during wintertime.

7.2 Stockpile materials bagged in flexible container

The new mixture needs to be sealed in an airtight bag for long term storage as it starts to react even with the moisture in the atmosphere. Small amounts such as 20 kg can be sealed in airtight bags but large amounts like 1 t are difficult and cannot be stored over long periods of time. Since mixtures harden as a result of the mixture and chemical reaction of the special lubricating oil, reaction assisting material, and water, as long as one of these is not present, the mixture can be stored semi-permanently. Consequently, by manufacturing the mixture without adding the reaction assisting material, then transporting
the mixture to the site of use and mixing in the reaction assisting material just before paving work as shown in photo 13, large amounts can be stored in flexible container bags, etc. This method not only enables asphalt paving in regions that do not have asphalt mixing plants such as remote islands, but also allows the mixture to be brought to disaster sites from afar and to be stored there. This is considered a technique useful for the restoration work after earthquake disasters.

Currently, producing and paving tests are being conducted to verify the serviceability of this technique. A simple device for mixing reaction assisting material on site is also being developed in the aim to put it to practical application.

7.3 Decolorizing

The new mixture can also be applied to decolorized asphalt, as well as fieldstone and bagged colored cold asphalt mixtures. Examples of application are shown in photo 14 and photo 15. Presently, durability and weather resistance are being verified. Applications such as repair material for fieldstone and colored pavements, small-scale pavements such as parks, and normal households are being considered.

7.4 Cold slurry asphalt suited to minor repair work

The technology for the new mixture may also be applied to cold slurry asphalt mixtures. Adding a special lubricating oil to liquid asphalt and mixing in a binder with reduced viscosity, sand, and a filler can produce an asphalt mixture that has liquidity at ambient temperatures, which may be used to make a filler for the repair of running lamp pipes on airport runways or a repair material for the pavement of poured asphalt on steel plate decks. In the past, airport running lamps would be installed in the following process: the pavement was first cut in grooves; power cables were laid out; water-tight liquid asphalt was carried in as it is constantly stirred and heated to about 240°C in a vehicle specially made for this purpose; it is then poured manually into the grooves to fill them. Such special vehicles would require a minimum of 4 t to be heated and stirred in them, however, and for small-scale paving work such as that for airport running lamps which typically use only about 1 t of a mixture, this would mean a larger amount of excess mixture that had to be disposed of, resulting in cost increases.

With the new mixture, meanwhile, the main blend comes in in a small can of a 20-liter capacity, to which the add-in material and water is added on site, using a handheld mixer. This simplified process makes the manual pouring of the mixture less complicated. Photo 16 shows how the new mixture could be poured in in real life. Since the new mixture may be used in paving work at ambient temperature, it prevents paving failures from occurring even during winter and increases the water-tightness of pavement. Furthermore, the elimination of excesses and their disposal has helped cut down on cost.
8. SUMMARY OF RESULTS

The new mixture has the following features:
- Although it is a bagged cold asphalt mixture, it has the same durability as hot asphalt mixtures.
- Strength development is via chemical reaction and is thus faster than volatile cold asphalt mixtures, enabling early traffic release.
- Paving is possible even in rain and subzero temperatures.
- Applicable to all particle sizes of aggregates. (However, the porous type is limited to sidewalks.)
- Can be stored for a long time (about 6 months).
- Paving method is the same as conventional cold asphalt mixtures except for need for water spraying. No special paving machine is required.
- It may be used in lieu of a regular hot asphalt mixture.
- It may also be used as a warm-mix and an improved-workability asphalt mixture, which helps reduce CO$_2$ emission and improve the working conditions for workers (e.g. protection against heat stroke).
- It is suited for long-distance transportation, allowing for the use in asphalt paving in an area where there is no asphalt mixing plant, such as remote islands.
- It may be applied to making a cold slurry asphalt mixture, which is suitable for pouring-in applications.

9. CONCLUSION

The new mixture has greatly improved in terms of the development of strength and the durability of the finished pavement, which have been the challenges facing conventional cold asphalt mixtures. It has a level of strength comparable to that of hot asphalt mixtures and can be manufactured at lower temperatures, which allows paving work to be performed at ambient temperatures. It makes a repair material that contributes to the reduction of CO$_2$ emission and improved working conditions for workers engaged in paving work (e.g. protection against heat stroke).

In the future, efforts will be made to further increase the scope of application such as use as decolorizing agent of natural color pavements and color pavement, and application as repair material of porous asphalt pavements, etc.
Evaluation of the Rutting Performances of Pavements with Cold Central Plant Recycled (CCPR) Bases Under Accelerated Pavement Testing

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EXTENDED ABSTRACT:

In recent years, Cold Central-Plant Recycling (CCPR) has attracted lots of interest in pavement engineering because of its cost-effective and environment-friendly characteristics. CCPR is a technology that produces a pavement mixture from Reclaimed Asphalt Pavement (RAP) without heating the recycled pavement material, and up to 100% of RAP can be used in the recycling process. The material is still primarily used on secondary or low-traffic roads because of lack of reliable performance data that document its long-term performance, therefore transportation agencies are interested in assessing its life-cycle performance. To test the long-term performance of CCPR pavements, two lanes of the test beds of the Virginia Accelerated Pavement Testing (APT) program were designed with CCPR as a base material.

The objective of this study is to analyze the performances of a CCPR base with different surface mix thicknesses under APT. The APT data collection included the responses of strain gauges, pressure cells, and temperature sensors embedded at various depths, as well as a daily three-dimensional scan of the surface of the test pavement to monitor rutting development.

During the construction of the test lanes, FWD tests were conducted in order to keep track of the pavement properties as the construction advanced and try to provide a homogeneous foundation. In particular, the effectiveness of the foundation layer (21B aggregate base with geogrid reinforcements) was verified using equations defined by Rohde and Scullion. The equations, based on the thickness of the asphalt layer, predict the bedrock depth from the deflection data obtained with the FWD. It was possible to observe that the 21B aggregate base with geogrid reinforcements acted as a uniform rigid foundation.

The pavement was modelled using the software 3D-Move, the computed responses were compared with measured stresses and strains. 3D-Move allows the viscoelastic characterization of the asphalt layers and the analysis of the pavement response to moving loads. A comparison between the two lanes was carried out, checking the values of vertical pressure recorded by the pressure cells and the values of maximum shear stresses (computed through software). It was observed how the thickness of the surface mix impacts both the intensity of the shear stresses in the asphalt and the reduction of stresses in the unbound layers. It was possible to calibrate the software rutting models (from
NCHRP 1-37A) to produce relatively accurate predictions of the rutting development for both lanes. The calibrated model was used to predict the effect of the surface mix thickness and to simulate the behaviour for five different surface thicknesses with the same pavement structure and operating conditions described for the APT facility lanes. The model can thus, be used to analyze other pavement structures subject to variable traffic and environmental loadings, which would allow applying the APT results to improve the analysis and design of pavements containing CCPR bases.

REFERENCES


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<th>Properties and Performance of Permeable Friction Concrete Reinforced by Fibers</th>
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<tr>
<td>Cheng-Te LEE</td>
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KEYWORDS: permeable friction concrete, cellulosic fiber, pavement performance

ABSTRACT: This paper presents the findings of a research study to evaluate the effect of cellulosic fibers on the engineering properties and field performance of permeable friction concrete (PFC). A control asphalt mixture test section was constructed along with fiber-reinforced asphalt mixture test sites. Laboratory tests were conducted on laboratory prepared test samples. The laboratory tests included drain-down and Cantabro abrasion in addition to field performance evaluation. The experimental results demonstrated that the use of fibers improved the drain-down and enhanced the durability of the PFC mixtures. The use of fibers appeared to reduce the potential for drain-down and weight loss; more so than did polymer modification. Field surveys indicated that adding fibers to asphalt binders improved rutting resistance with the PFC mixed polymer-modified asphalt showing the best performance in the field. Performance was improved by the addition fibers to PFC according to measurements in the field. The use of fibers applied to permeable friction asphalt mixes was shown to be a viable option for pavement surface layers.
Properties and Performance of Permeable Friction Concrete
Reinforced by Fibers

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1 INTRODUCTION

Drainability is one of the main characteristics of permeable friction concrete (PFC) mixtures and is the primary reason for using these mixtures around the world (Donavan 2014; Lyons and Putman 2013; Shirke and Shuler 2009). PFC is an open-graded mixture placed at the surface of asphalt pavements to produce benefits in terms of safety, economy, and the environment. The air voids of PFC are created by gap grading the coarse aggregates and either eliminating or minimizing the volume of fine aggregates in the mixture to form a network of interconnected pores within the material. PFC is an environmentally friendly road material using advanced technology in pavement design. PFC applied to the surface layers usually has an air void content of approximately 20 percent. Due to higher proportions of coarse aggregates and lower sand content, interconnected air voids are created which, in wet weather, drain the moisture through a series of water channels inside PFC. This drainage system prevents aquaplaning on the road surface and improves visibility. The high porosity of PFC also reduces traffic induced noise emissions (Huurman et al., 2010; Masad et al., 2004).

Although the use of PFC mixtures provides various advantages due to the high air voids content and the large permeability, PFC can also pose some disadvantages, such as reduced performance, high construction costs, winter maintenance problems, and limited structural contribution (Bendtsen and Andersen 2004; Suresha et al., 2010). Reduced performance of PFC mixtures is associated with reduced durability and functionality (i.e., drainability and noise reduction effectiveness), due to raveling and clogging, respectively. In addition, for pavement structure design, PFC mixtures are typically considered to have limited structural contribution (Estakhri et al., 2008; Martin et al., 2014).

To maintain the benefits of a PFC layer, a mix design system that produces mixtures that are both functional and durable is required. Functionality of PFC mixtures includes properties related to drainability and surface friction. Drainability and noise reduction effectiveness are the main functional properties of PFC mixtures that justify using these mixtures as surface layers in asphalt pavements (Fwa et al., 1999; Masad et al., 2004). However, raveling is the distress most frequently reported as the cause of failure in PFC mixtures (Isenring et al., 1990; Sprinkel et al., 2015). The engineering properties of PFC mixtures could be lower than those of the conventional dense-graded asphalt mixtures. High air voids created inside PFC mixtures could lead to a potential loss of cohesion and less resistance to disintegration in the mix.

Fiber reinforcements have been used for decades in Portland cement concrete as well as in asphalt concrete mixes (Abtahi et al., 2010; Stemphar et al., 2012; Mohammed et al., 2018). Technologies that use fibers in Portland cement concrete mixes have been investigated widely since the 1950s. Fibers could also play a significant role to mitigate distresses in asphalt concrete and to increase its strength. Nonetheless, research has been limited on the use of fibers to improve the performance of hot-mix asphalt (HMA) for pavement applications. McDaniel (2015) recently summarized the state of practice of the use of fibers in asphalt pavements in a synthesis published by National Coordinated Highway Research Program (NCHRP). The report indicated that most states have used fibers in open-graded mixtures. Researchers have used fibers in dense-graded asphalt mixes (Kaloush et al., 2010; Gibson and Li, 2015; Klinskyet al., 2018). The types of fibers used in asphalt mixtures included mineral, glass, cellulose, and synthetic polymer fiber. The mix design procedure for fiber was found to be the same as it was for conventional mixes. However, results have been inconsistent about the benefits of fibers. In some studies, fibers improved mix resistance to rutting and cracking. In others, no significant difference was observed in the fiber-reinforced mixes.
Both polymer-modified asphalt binders and fibers are the common methods to improve the performance of permeable friction course. The effects of all these materials could be two-sided: when they improve the mixture’s performance in one aspect, they might decrease its performance in the other aspect. There is a need to investigate the functionality and the durability of PFC mixtures in the long run. This paper presents a laboratory investigation conducted in conjunction with a pilot project in which experimentation with the use of fibers took place in a field test. The goal of the project was to determine whether the fibers could improve pavement performance at the location of the project site. To achieve this goal, the pilot project included two phases, a laboratory evaluation phase and a field evaluation phase. The main objective of this study focuses on evaluating the engineering properties of PFC mixtures, and assessing the PFC performance according to field measurements.

2 MATERIALS AND MIXTURES

Asphalt binders commonly used in Taiwan for PFC pavements were included in this study as follows: conventional bitumen (CB), and polymer-modified bitumen (PB). While CB was unmodified, PB was a particularly-manufactured binder represented by a relatively high viscosity as listed in Table 1. The mixing and compaction temperatures for the CB binder were selected corresponding to 0.17 Pa.s and 0.28 Pa.s viscosities, respectively. For polymer-modified asphalt, the mixing and rolling temperatures were recommended by manufactures.

Table 1. Asphalt Properties

<table>
<thead>
<tr>
<th>Properties</th>
<th>CB</th>
<th>PB</th>
</tr>
</thead>
<tbody>
<tr>
<td>Penetration (25°C, 0.1 mm)</td>
<td>65</td>
<td>56</td>
</tr>
<tr>
<td>Softening point (°C)</td>
<td>49</td>
<td>65</td>
</tr>
<tr>
<td>Solubility (%)</td>
<td>99</td>
<td>99</td>
</tr>
<tr>
<td>Viscosity (60°C, poise)</td>
<td>3,200</td>
<td>15,600</td>
</tr>
<tr>
<td>RTFOT Viscosity (60°C, poise)</td>
<td>8,900</td>
<td>20,800</td>
</tr>
<tr>
<td>Penetration ratio (25°C, %)</td>
<td>66</td>
<td>72</td>
</tr>
<tr>
<td>Loss of mass (%)</td>
<td>0.01</td>
<td>0.02</td>
</tr>
</tbody>
</table>

Cellulose fibers shown in Figure 1(a) had a maximum length of approximately 6 mm. They were subjected to sieve analysis, ash content, pH, oil absorption, and moisture content tests for the fibers used in permeable friction concrete. The results of these tests are shown in Table 2. More than 70% of the fibers were retained on the 850 mm sieve. The ash content as a percentage of nonvolatiles was low compared with the specification percentage. The pH results indicated that the fibers are neutral. The oil absorption of the fibers was 3.6 times their weight and the moisture content was 4.5%. A micrograph of the fibers is given in Figure 1(b) using a scanning electron microscope (SEM). Cellulose fibers could form a three directional network in bitumen, which could be interacted with bitumen when subjected to loadings and high temperatures.

Table 2. Fiber Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Fiber</th>
<th>Specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length, mm</td>
<td></td>
<td>Max. 6</td>
</tr>
<tr>
<td>Sieve analysis</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Passing 850 mm</td>
<td>28.6</td>
<td>85 ± 10%</td>
</tr>
<tr>
<td>Passing 425 mm</td>
<td>20.2</td>
<td>65 ± 10%</td>
</tr>
<tr>
<td>Passing 0.106 mm</td>
<td>16.8</td>
<td>30 ± 10%</td>
</tr>
<tr>
<td>Ash content, % nonvolatile</td>
<td>5.8</td>
<td>18 ± 5</td>
</tr>
<tr>
<td>pH (testing temperature)</td>
<td>7.25 (20.9°C)</td>
<td>7.5 ± 1.0</td>
</tr>
<tr>
<td>Oil absorption, number of times its own weight</td>
<td>3.6</td>
<td>5.0 ± 1.0</td>
</tr>
<tr>
<td>Moisture content, %</td>
<td>4.5</td>
<td>Max. 5</td>
</tr>
</tbody>
</table>
Fibers were added to the asphalt mix in a manner that insures complete blending of the fiber with the aggregates and liquid asphalt binder. In a batch plant the fiber was added into the weigh hopper as shown in Figure 2(a), and simultaneously blown to the hot aggregate in a dry-mixing procedure. These fibers were added at 0.3% by weight of total mix. When the fibers were furnished in uniformly weighed containers, the fibers were then added directly to the pugmill shown in Figure 2(b). Dry mixing time was increased five seconds to insure adequate blending. Wet mixing time was increased at least five seconds for cellulose fibers to help ensure uniform distribution.

Coarse aggregate for a PFC mix must be strong to carry the imposed loads because coarse aggregate plays an important role in carrying the traffic loads for PFC mixtures. The properties of limestone coarse and fine aggregate were summarized in Table 3. The LA abrasion value of coarse aggregate should be less than 30% to possess sufficient toughness. In addition, flat and elongated proportions must be limited to be a minimum value, and fracture faces are required to provide a coarse aggregate structure with high internal friction.
 souls is each of the following two types of asphalt binders allowed for PFC mixtures: CB and PB. CB and PB mixtures need to be mixed with a minimum of 0.3% cellulose fibers by weight of total mix. The 19-mm maximum aggregate size gradation was gapped on the 4.75-mm sieve. The percentage of mineral fillers was approximately 5 percent. Open-graded mixtures that are designed to have a minimum of 18 percent air voids are considered PFC. The target air void content of 20 percent was selected to enhance environmental benefits of PFC mixtures. The job mix formula was decided using the Marshall mix design method with specimens prepared by applying 50 blows on each faces. The range of allowable binder content was determined using the following two properties: abrasion and drain-down tests. The asphalt binder content was decided to be 5.0% for all PFC mixtures.

<table>
<thead>
<tr>
<th>Types</th>
<th>Properties</th>
<th>Test results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse Aggregate</td>
<td>LA abrasion (%)</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>Soundness (%)</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Flat &amp; elongated (%) 3:1</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5:1</td>
</tr>
<tr>
<td></td>
<td>Absorption (%)</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td>Fracture faces (%) one</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>two</td>
</tr>
<tr>
<td>Fine Aggregate</td>
<td>Soundness (%)</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Sand equivalent value (%)</td>
<td>82</td>
</tr>
</tbody>
</table>

Aggregate gradation listed in Table 4 is for each of the following two types of asphalt binders allowed for PFC mixtures: CB and PB. CB and PB mixtures need to be mixed with a minimum of 0.3% cellulose fibers by weight of total mix. The 19-mm maximum aggregate size gradation was gapped on the 4.75-mm sieve. The percentage of mineral fillers was approximately 5 percent. Open-graded mixtures that are designed to have a minimum of 18 percent air voids are considered PFC. The target air void content of 20 percent was selected to enhance environmental benefits of PFC mixtures. The job mix formula was decided using the Marshall mix design method with specimens prepared by applying 50 blows on each faces. The range of allowable binder content was determined using the following two properties: abrasion and drain-down tests. The asphalt binder content was decided to be 5.0% for all PFC mixtures.

<table>
<thead>
<tr>
<th>Results</th>
<th>Sieve Size (mm)</th>
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<tbody>
<tr>
<td>Specification</td>
<td>19</td>
</tr>
<tr>
<td></td>
<td>12.5</td>
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<tr>
<td></td>
<td>4.75</td>
</tr>
<tr>
<td></td>
<td>2.36</td>
</tr>
<tr>
<td></td>
<td>0.075</td>
</tr>
<tr>
<td>Passing (%)</td>
<td>100-95</td>
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<tr>
<td></td>
<td>84-64</td>
</tr>
<tr>
<td></td>
<td>31-10</td>
</tr>
<tr>
<td></td>
<td>20-10</td>
</tr>
<tr>
<td></td>
<td>7-3</td>
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<tr>
<td></td>
<td>98</td>
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<td>66</td>
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<td>15</td>
</tr>
<tr>
<td></td>
<td>4.8</td>
</tr>
</tbody>
</table>

3 LABORATORY TESTS

Laboratory tests including drain-down and durability were conducted to investigate the engineering properties of PFC mixtures. The testing data presented in this study are the average values of three replicates.

3.1. Drain-Down

The binder drainage test, combined with determination of volumetric properties, was used to evaluate the potential of binders draining down from the aggregate in accordance with ASTM D6390. In addition, this test could be used to determine the target binder content for PFC mixtures to maximize binder content so as to optimize durability while eliminating possible binder drainage. A sample of loose asphalt mixture to be tested was prepared and placed in a wire basket. The sample and the container were heated in a forced draft oven for one hour at 165°C. At the end of one hour, the basket containing the sample was removed from the oven along with the container and the mass of the plate or container was determined. The amount of drain-down was then calculated.

Figure 3 indicates that the drain-down increased with increasing the binder content. Each bar represents the average of three tests with the variation ticks indicating one standard deviation of the tests. A maximum drain-down of 0.3% by weight of total mix is used as the limiting value for determining acceptable performance. PFC mixtures without fibers have significantly more drain-down. At 0.3% cellulose fibers by weight of mixture, drain-down was reduced to be about 0.1% at the asphalt content of 5%. This suggests that
fibers help hold the asphalt binder to the aggregate structure of a porous asphalt mix. The amount of binder loss of the PB mix was also lower than that of the CB mix. Reduction in drain-down could be attributed to the increase in the viscosity of polymer-modified asphalt as well as the addition of fibers. The use of fibers appears to greatly reduce the potential for drain-down; more so than did polymer modification as shown in Figure 3.

![Figure 3. Laboratory results of drain-down test with different binder contents.](image)

3.2. Durability

The durability of asphalt mixtures may be regarded as resistance to damage caused by environmental factors. In simple terms mix durability is the ability of a mix to perform. The ability to maintain structural integrity may also be related to the cohesiveness of the mixture. The Cantabro test has been used to determine the cohesiveness of the PFC mixtures. The test method entailed compacting a PFC mix compacted by 50 blows on each site at the optimum asphalt content, allowing the specimen to cool to a test temperature of 25°C, weighing the specimen to the nearest 0.1 g, and then placing the specimen into a Los Angeles Abrasion machine without the steel balls used in the test. The Los Angles Abrasion machine was operated for 300 revolutions at a rate of 30 rpm. After 300 revolutions, the specimen is removed and again weighed to the nearest 0.1g. The change in mass before and after testing is an indication of the durability of the mixture. The lower the weight loss is, the better the durability.

Figure 4 shows that the CB mix had the highest abrasion loss, and the PB+fiber mix had the lowest abrasion loss. A maximum weight loss of 20% is specified for PFC. As compared with the CB binder, the use of fibers reduced the weight loss by 35% to 15%. The use of polymer-modified asphalt is also shown to make a difference in results for the Cantabro weight loss test. According to drain-down and durability test results, three binder types, i.e., PB, PB+fiber and CB+fiber, were selected for the construction of a test road.
4 FIELD PERFORMANCE

The site selected was on an urban roadway in central Taiwan, a distance of approximately 3 km. The highway department identified this location to address pavement distresses and safety on this road, heavily trafficked by truck traffic. The project involved milling and overlaying 10 cm (4 in.) of the existing roadway and replacing it with fiber-reinforced PFC in two sections. A polymer-modified third section was used as a control. The three sections were approximately equal in length. Construction was completed in January 2010. The PFC pavements with PB, CB+fiber, and PB+fiber test sections are located adjacent to each other with no exits among them. Monitoring in-service pavements is one of the best methods for gaining data on how fiber-reinforced PFC pavements will perform over time under real environmental and traffic conditions. Surveys for pavement performance including drainability, rutting, and friction were conducted at scheduled intervals.

4.1. Drainability

The Public Construction Commission (PCC) in Taiwan specifies a minimum field drainability value, measured in terms of specific water volume within an outflow time. The outflow time of a specific water volume can be considered as an indication of the mean rate of water discharge. Although these measurements are often referred to as field permeability, the outflow time cannot be used to calculate the coefficient of permeability since the area and direction of flow during the test are not controlled. Nevertheless, the outflow time is a useful parameter to compare the performance in terms of drainability for different mixtures.

Detailed examination of drainability is warranted, since drainability is one of the main characteristics of PFC mixtures and closely related to their advantages. A field drainability device made of a Plexiglas cylinder connected to a steel base was used for measuring drainability of the porous asphalt layer as shown in Figure 5. This Plexiglas cylinder has an interior diameter of 50.48 mm and a height of 350 mm. The Plexiglas cylinder contains engraved markings that are 100 ml apart, with the “zero” marking being a height of 600 mm above the pavement surface. Special clay was placed as a 30 mm wide ring on the pavement surface in order to cover the highest aggregate and fill voids at the surface besides an O-ring. Thus the contact zone between the permeameter and the pavement was sealed, and the water was forced to flow through the interior voids of the porous asphalt layer. Permeability of the layer is expressed as the time elapsed between the 100 and the 500 ml line. The downward movement of the water corresponds to an outflow quantity of 400 ml. When the time needed to pass the 900 ml volume is less than 15 sec, the drainability of the permeable friction course is considered to be sufficient.
Figure 5. Field measurement of drainability.

Figure 6 shows the drainability of the permeable friction wearing course was almost the same in the beginning, but decreases over time in general. Each bar represents the average value of three replicate tests, and the tick indicates one standard deviation from the average value. The reduction rate appears to be binder and fiber specific with the PB+fiber section having the best drainability. The PB test section had lower drainability as compared with the CB+fiber and PB+fiber sections. After 88 months in service, a slight filling up of the PFC layer occurred for all the test sections. The water evacuation time measured with the field permeameter had gone from 1,399 to 1,180 ml per 15 seconds. The addition of fibers is shown to help maintain drainability. The reason might have been linked to the additional strength of fibers provided. Increased strength enhanced the ability of asphalt attached to PFC without drain-down, thus preventing binder clogging during summer time. The slight reduction had no appreciable effects on the efficient of water draining and the avoidance of splashing. Besides, no raveling, cracking or other failures have been observed on the test sections to any significant extent since they were open to traffic.

![Figure 6. Field results of drainability measurements (mo=month).](image-url)
4.2. Rut Depth

Rut depth was measured in the left and right wheel paths with a 1.2-m straightedge according to ASTM E1703. The results of the rut depth are summarized in Figure 7. The severity level is considered to be low when the mean rut depth is less than 12.5 mm, moderate when rutting is between 12.5 and 25 mm, and high when rutting is higher than 25 mm. The severity level of these three test sections was shown to be low after 88 months in service. The low rutting values indicate that the PFC layer possesses good resistance to plastic deformation since PFC has a coarse gradation that results in stone-on-stone contact. Rut depth ranged from 4.0 mm for the PB+fiber section to 7.1 mm for the CB+fiber section. The rut depth of the CB+fiber section was highest among the three test sections, that of the PB+fiber section the lowest, and that of the PB section in between. The polymer-modified binder appears to more effectively reduce rutting compared with fibers. Nevertheless, the addition of fibers to polymer-modified asphalt contributes to the reduction in permanent deformation. Other factors are clearly important for controlling pavement rutting. The aggregate skeleton, overall mix design, and construction quality have a profound effect on rutting resistance, and all of them played a key role in this study.

![Figure 7. Field results of rutting measurements.](image)

4.3. Friction

The British Pendulum Tester was performed to measure pavement skid resistance according to ASTM E303. The British Pendulum number (BPN) values were all adjusted by the exact pavement temperature at measurement to an equivalent BPN value at 20°C. The measurement of the friction showed an initial BPN of 61 to 62 as demonstrated in Figure 8. Skid resistance was relatively low just after construction because of asphalt binder film coating the aggregate at the pavement surface. As a consequence of the disappearance of the binder film covering the surface of the aggregate, skid resistance was improved after PFC open to traffic. After twelve months, it was already 64 and after twenty four months 65. A BPN of 45 is considered sufficient and safe for highway pavements. Test results in Figure 8 indicate that PFC layers provided good wet weather friction. The BPN value does not appear to be correlated with the fiber added to the PFC mixture. According to field surveys, the positive features of PFC mixes included good frictional properties and high drainability after 88 months in service. Pavement skid resistance is dependent on microtexture and macrotexture that are more related to aggregate as compared with fiber.
5. CONCLUSIONS AND RECOMMENDATIONS

This paper is to evaluate the engineering properties of permeable friction concrete (PFC) in laboratory, and compare the field performance of a test road consisting of three test sections. Conclusions subsequently drawn are based on the analysis of limited testing results obtained from laboratory and field evaluations.

- Mixtures without fibers or polymers showed greater drain-down than those with additives. The use of fibers made it easier to obtain a greater resistance to drain-down, particularly for the conventional mix.
- The weight losses in the Cantabro test were lower when using fibers compared with that without fibers. The combined use of polymer modified binder and fiber minimized the abrasion loss and, thus, increased the durability of PFC mixtures.
- During the construction of the test sections, it was essential that the fibers be well distributed in the mix. For this reason, it was critical to monitor the distribution of the fibers during the production.
- Rutting resistance of permeable friction asphalt mixes, as measured by field performance, was improved using polymer-modified asphalt as well as fibers.
- The drainability of the fiber-reinforced asphalt sections was higher than that of the traditional asphalt section.
- PFC pavements provided good wet weather friction resistance once the asphalt binder film worn from the aggregate. Pavement friction was more associated with aggregate than fiber.
- Test results obtained from both the laboratory and the field suggest that the use of fibers applied to permeable friction asphalt mixes be a viable option for pavement surface layers.

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Effect of Binder Content and Active Filler on Foamed Bitumen Mixtures for Cold Foamed In-Place Pavement Recycling Technology in Uganda

PAPER TITLE

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RAP, Foamed Bitumen, Active filler, Stabilisation, Rehabilitation

ABSTRACT:
The recycling of asphalt pavements has become a common practice in the transportation industry worldwide. Pavement recycling is especially very effective for rehabilitating existing pavements or constructing new pavements while reducing construction costs, environmental impacts, and construction time. The utilization of Reclaimed Asphalt Pavement (RAP) in highway construction is in line with the global target of utilization of natural resources in such a way that promotes sustainable development. In Uganda the use of RAP in asphalt production, though being in early stages and still underdeveloped, has been undertaken on a selected few national road rehabilitation projects and is earmarked for other future projects. Utilization of recycled materials is an essential component to the long term sustainability of Uganda’s pavement networks since the country is currently undertaking massive investments in road infrastructure development projects aiming to develop, expand and increase the capacity of the old roads and maintain the new roads in the road network.

This study is aimed at coming up with a suitable laboratory mixture design for foamed bitumen for rehabilitation of road pavements through the use of cold in-place recycling technology. This is part of the wider project on the development of technical specifications for cold foamed in-place pavement recycling technology in Uganda. The study investigates the effect of foamed bitumen and active filler (cement) on the mechanical properties cold recycled base course materials.
Effect of binder content and active filler on foamed bitumen mixtures for cold foamed in-place pavement recycling technology in Uganda

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1. INTRODUCTION

The recycling of asphalt pavements has become a common practice in the transport sector worldwide. The utilisation of Reclaimed Asphalt Pavement (RAP) has been in practise since 1930s and just like any other materials, the principal factors behind the efforts worldwide include reduction of construction waste, preservation of non-renewable natural resources, and lower energy costs. The use of RAP in roadway construction fits with the global objective of sustainable development by the prudent use of natural resources. RAP recycling activities address the issues of reducing use of declining virgin aggregate sources, material storage and disposal of reclaimed asphalt material from paving projects. Further, energy savings can be realized through the utilisation of RAP in roadway construction by reducing the processing and haulage of virgin aggregate materials. Other factors include potentially faster project completion and reduction in construction-related traffic, resulting in less disruption to the traveling public (Jitendra and Jie 2015, Federal Highway Association 2011).

In recent years, the Government of Uganda through Uganda National Roads Authority (UNRA) has embarked on programs to revitalize and develop the national road infrastructure. These programs have involved massive investments in road construction and rehabilitation projects aiming to develop, expand and increase the capacity of the old roads and maintain the new roads in the road network. Asphalt pavements that have reached the end of their service life are undergoing rehabilitation by milling the existing pavement surfaces and replacing the milled portion with new Hot Mix Asphalt (HMA). A large amount of recycled asphalt pavement (RAP) is and will continue being generated every year because of this practice (Uganda National Roads Authority).

In order to deal with the large amounts of RAP expected in the future as well as take advantage of the economic savings and environmental benefits of using recycled materials, a study to develop foamed bitumen mixtures for cold foamed in-place pavement recycling technology for rehabilitation of existing pavements in Uganda was initiated. Foamed Bitumen In-placed Recycling (FBIR) is the process that uses reclaimed material from distressed pavement along with bitumen in the form of foam known as foamed bitumen. When small quantity of cold water (2 to 5%) is injected into a hot bitumen at a temperature of 160°C-180°C, the hot bitumen expands approximately 15-20 times to its original volume and forms a foam. In this foamed state, bitumen will have very large surface area and an extremely low viscosity. The produced foamed bitumen contains thousands of bitumen bubbles, which is then ready to mix with aggregates. The technique is called “cold” because the technique does not require heating of the existing pavement or reclaimed material as in conventional ‘hot’ recycling techniques and thus requires less energy than other pavement recycling methods. Foam bitumen treatment is preferably utilised in a base layer, and therefore the surface course must be provided over the stabilized layer (Asphalt Academy 2009).

Foamed Bitumen In-situ Recycling (FBIR) is therefore an innovative technique where the structural capacity of a distressed pavement can be improved using existing pavement materials with some addition of fresh materials. This technique is being adopted in a number of countries and extensive research and attempts to standardise it has been conducted in many parts of the world including South Africa, Australia, India, UK etc. (Gonzalez 2009, Wirtgen 2004, Jenkins 2000, Fu 2010, Jones 2009, Wirtgen 2010). Studies have shown that RAP for use in base and subbase layers can be characterized by performance-related parameters and properties including those needed for pavement design, such as grading, resilient modulus, shear strength
under static triaxial loading, and permanent deformation under repeated triaxial loading, and those identifying material durability, such as frost susceptibility and abrasion resistance (Virginia Center for Transportation Innovation and Research 2015).

Though the technology is gaining popularity in Uganda, it has only been implemented on a selected few roads in Uganda such as the Jinja – Bugiri rehabilitation project (Uganda National Roads Authority 2011). This is largely because no guidelines and/or specifications for the technology have been developed, which are tailored to the local conditions. The increasing use of FBIR technique in Uganda has necessitated the development of guidelines for selection of suitable project sites, standard mix design procedures, appropriate structural design approach and correct construction practices. The current research is therefore aimed at coming up with a suitable laboratory mixture design for foamed bitumen. It’s mainly focused on the effect of active filler (cement) and foamed bitumen on the mechanical properties cold recycled base course materials. This is part of the wider research project on the development of technical specifications and guidelines for cold foamed in-place pavement recycling technology in Uganda.

2. MATERIALS AND RESEARCH APPROACH

2.1 Foamed bitumen characteristics

In this research a 80/100 penetration grade virgin bitumen with a 102 dmm penetration at 25°C, softening point of 54.6°C and dynamic viscosity at 60 °C of 0.22 Pa.s was used. This meets the criteria specified by (Asphalt Academy 2009) and (Wirtgen 2004), which recommend a minimum penetration value of 80 dmm to be used for foaming, while corresponding maximum values are limited to 100 dmm and 150 dmm respectively to avoid poor stability of the mixtures. Harder grades of bitumen are usually not recommended because they tend to give poor foam characteristics and only modest dispersion in the mixture.

A laboratory-scale foamed bitumen machine, Wirtgen WLB 10S, was used to produce the foamed bitumen. When 3.0 % reaction water was incorporated with hot bitumen at roughly 180°C, air pressure of 4 bars and 0.4% foaming agent (INTERFOAM B), a foamed bitumen product with an expansion rate of 14 times and a half-life of 31.9 seconds was yielded. It was deemed to be a good foam quality since the foaming characteristics were within those specified by different agencies (Asphalt Academy 2009, AARA 2017, Austroads Ltd 2013).

2.2 Aggregate properties

Virgin aggregates

The virgin mineral aggregate used was crushed limestone obtained from a local quarry. Tests were conducted on virgin aggregate to determine its mechanical properties. Aggregate Impact Value of 21.2 %, Water absorption of 0.58 %, Abrasion test of 21.5 %, Aggregate crushing value of 21.5 % and 10% fines value of 186 kN were obtained. Sieve analysis was also conducted on the aggregates and details of the gradation of the virgin mineral aggregate are given in Table 1.

| Table 1: Gradation of aggregates |
|-------------------------------|---|---|---|---|---|---|---|---|---|
| Percentage passing (%)        | 100 | 98.6 | 76 | 59 | 46 | 33 | 24 | 18 | 12 | 8 | 3.5 |
| Sieve size (mm)               | 28.0 | 20.0 | 14.0 | 10.0 | 5.00 | 2.36 | 1.18 | 0.600 | 0.300 | 0.150 | 0.075 |
| RAP aggregate                | Virgin aggregate |

RAP aggregate

RAP aggregate material used was obtained from Kampala-Masaka road comprising of thick asphalt concrete and base course materials. Samples were milled from 20 different locations and combined in order to ensure that there was an adequate amount for conducting laboratory foam bitumen mix design. Bitumen was extracted from the collected RAP material and Penetration, Softening point and Viscosity tests were conducted. The values of 100 dmm penetration at 25°C, softening point of 46.6°C and dynamic viscosity at 60 °C of 0.29 Pa.s were obtained. The amount of bitumen recovered from the aggregate was found to be
4.8%. Sieve analysis was conducted on the RAP aggregate and the details of the gradation of the RAP material are given in Table 1 above.

According to the design, the constituent material for the untreated material will incorporate 50 mm thickness obtained from existing road under consideration (RAP material) and an additional crushed stone (virgin aggregates) in varying amounts from 120mm to 160mm. Thus, the laboratory mix design of foamed bitumen was conducted by blending 25.4% (50mm) of the existing road material (RAP) with 74.5% of the crushed stone (140mm).

2.3 Optimum Compaction Moisture content (OCMC)

The optimisation of compaction moisture content was carried out on specimens compacted using a vibrating hammer to densities that were obtained by modified proctor compaction. The OCMC normally approximates the optimum moisture content (OMC) of the untreated material. According to (Asphalt Academy 2009), these values range from 60 to 90% of the OMC determined by modified proctor compaction for the untreated material. An analysis in order to determine the percentage of water required to achieve the maximum dry density was carried out in the laboratory. An optimum moisture content (OMC) and Maximum dry density (MDD) of 3.9% and 2190 kg/m$^3$ respectively, were determined as using the moisture – density relationship. These values were used in the mix design.

2.4 Mixing and compaction

Mixes of varying ingredients of active filler (cement) and bitumen content were prepared using a pug mill type of mixer, compacted and cured. Active filler content of 0.5% was initially held constant whilst the binder content was varied in ranges of 1.75%, 2.0% and 2.5%. Specimens were molded from each proportion of binder content mix, cured and then tested for their mechanical properties of Indirect Tensile Strength in the dry (ITS-Dry) and wet (ITS-Wet) condition, TSR (Tensile Strength Ratio) and Resilient modulus (MR). The same procedure was repeated when varying the active filler (cement) content to 0.75% and 1.0% in order to come up with the optimum percentage of active filler and binder content to be used. Addition of active filler (such as cement or hydrated lime) is recommended for a number of reasons such as to improve resistance to water, to achieve the required amount of filler or to increase early strength so as to accommodate early traffic. However, it is recommended that the use of cement in excess of 1.5% by mass should be avoided due to the negative effect on the flexibility of the stabilised layer (Wirtgen 2004). Furthermore, (Asphalt Academy 2009) suggests the use of cement and hydrated lime as active fillers, but recommends that when cement is added as the active filler the amount should be limited to a maximum of 1% by mass of dry aggregate.

3 RESULTS AND DISCUSSION

3.1 Foam Bitumen Mix Design

<table>
<thead>
<tr>
<th>Mix Design procedure</th>
<th>Age (Days)</th>
<th>Reaction water (%)</th>
<th>Foaming Agent (%)</th>
<th>Binder content (%)</th>
<th>Active filler % (Cement)</th>
<th>Expansion Ratio (Times)</th>
<th>Expansion Ratio after 15 seconds (mm)</th>
<th>Half Life (Sec)</th>
<th>Mixing Moisture (%)</th>
<th>MDD (kg/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trial 1</td>
<td>7</td>
<td>3.0</td>
<td>0.4</td>
<td>1.75</td>
<td>0.5</td>
<td>19.2</td>
<td>13.3</td>
<td>31.9</td>
<td>3.9</td>
<td>2190</td>
</tr>
<tr>
<td>Trial 2</td>
<td>7</td>
<td>3.0</td>
<td>0.4</td>
<td>1.75</td>
<td>0.75</td>
<td>19.2</td>
<td>13.3</td>
<td>31.9</td>
<td>3.9</td>
<td>2190</td>
</tr>
<tr>
<td>Trial 3</td>
<td>7</td>
<td>3.0</td>
<td>0.4</td>
<td>1.75</td>
<td>1.0</td>
<td>19.2</td>
<td>13.3</td>
<td>31.9</td>
<td>3.9</td>
<td>2190</td>
</tr>
</tbody>
</table>
This experimental study was conducted of foam bitumen with varying percentages of active filler and bitumen content. The foaming characteristics of expansion ratio of 19.0 and half-life of 31.9 seconds which were obtained in Section 2.1 were used and are within those specified by (Asphalt Academy 2009). The study was aimed at evaluating the effect of active filler (cement) and foamed bitumen on the mechanical properties cold recycled base course materials in order to come up with an optimal active filler and bitumen content.

Foam Bitumen Mix Design was carried out by varying the active filler at percentages of 0.5, 0.75 and 1.0 %. For each of the percentage of active filler, bitumen content was further varied at percentages of 1.75, 2.0 and 2.5 %, as shown in Table 2 above. The reaction water, mixing moisture and foaming agent (INTERFOAM B) were fixed for all the mixes at 3.0 %, 3.9 % and 0.4 % respectively. The mechanical properties (ITS-Dry, ITS-Wet, TSR and M_r) for the different mixes were carried out on the specimens compacted using a vibrating hammer after a 7 days curing period.

3.2 Effect of active filler

For all the mechanical properties tested, it was clearly observed that 0.5 % of active filler resulted in low strength and stiffness values as shown in Figure 1 to Figure 5.
Figure 2. Relationship between ITS–Wet and bitumen content at different percentages of active filler

Figure 3. Relationship between TSR and bitumen content at different percentages of active filler
Figure 4. Relationship between $M_R$-Wet and bitumen content at different percentages of active filler

Figure 5. Relationship between $M_R$-Dry and bitumen content at different percentages of active filler

It was generally observed that at all bitumen contents, all the mechanical properties increased with increasing amount of active filler content as shown in Fig.1 – 5. However, it was importantly observed that the increment in the mechanical properties was more pronounced when the active filler content was increased from 0.5 % to 0.75 % than when increased from 0.75 % to 1.0 %. For example, it can be observed in Fig.1 – 3 that when the active filler was increased from 0.5% to 0.75% for a fixed bitumen content of 2.0%, the ITS-Dry, ITS-Wet, TSR, $M_R$-Dry and $M_R$-Wet increased by 46 %, 97%, 35%, 43% and 88% respectively. However when the active filler was increase from 0.75% to 1.0% for the same fixed bitumen content of 2.0%, the ITS-Dry, ITS-Wet, TSR, $M_R$-Dry and $M_R$-Wet increased by only a negligible amount of 1.5 %, 2.5%,
1.0%, 1.5% and 2.3% respectively. Similar trends were observed for the other bitumen contents of 1.75% and 2.5%. It was further observed that for 0.5% of active filler at all bitumen contents, the ITS-Wet and TSR values did not the minimum values recommended. (Asphalt Academy 2009) and (Wirtgen 2010) recommends minimum values for ITS-Dry and ITS-Wet of 225 kPa and 100 kPa respectively.

Generally, it can be observed that addition of active filler plays a key role in increasing the stiffness of the mixes as well as the rate of strength gain since all mixes met the minimum values recommended after a 7-day curing period. (Wirtgen 2010) recommends that the resilient modulus values of BSMs should range between 1000 – 2000 MPa when 100% RAP material is used for bitumen content ranging between 1.6 to 2.0%. It further recommends that the resilient modulus values should range between 800 – 1500 MPa when a combined RAP/Crushed stone (50:50 blend) material is used for bitumen content ranging between 1.8 to 2.5%. The current study utilized a 25:75% (RAP/Crushed stone) blend of the material and the achieved stiffness values are within the ranges specified above. However, it was observed that a low percentage of 0.5% of active filler is not adequate for the mixes to meet the required strength requirements and therefore a 1.0% active filler content is recommended. This is in agreement with (Asphalt Academy 2009) which limit the amount of active filler (cement) to a maximum of 1.0% and emphasizes that beyond 1.5% by mass, cement has a negative effect on the flexibility of the stabilized layer.

3.3 Effect of Bitumen content

The effect of bitumen content was more pronounced at a lower percentage of active filler of 0.5%. It was observed that at 0.5% of active filler all the strength and stiffness values increased with increasing bitumen content from 1.75% to 2.0%. The peak values were observed at 2.0% bitumen content. Thereafter there was a drop in the values when the bitumen content was increased from 2.0% to 2.5%. A similar but less pronounced impact of the bitumen content was observed for 0.75% of active filler with the strength and stiffness values generally following a similar trend as observed for 0.5%. However, there was no impact on strength and stiffness values at 1.0% of active filler when the bitumen content was varied. Therefore a bitumen content of 2.0% was determined as the optimal value at all the strength and stiffness values. Again, it has been demonstrated that the amount of active filler incorporated in the mix is the overriding factor since it will affect the adhesion of the bitumen to the aggregates as well as affecting the dispersion of the bitumen in the mix (Wirtgen 2010). Therefore an optimal value of bitumen content should be obtained using the optimal active filler content by varying different percentages of bitumen content.

4. CONCLUSION

This paper presented an experimental study on the effect of active filler (cement) and foamed bitumen on the mechanical properties cold recycled base course materials. The following conclusions are drawn based on the experimental findings:
- Addition of active filler plays a key role in increasing the stiffness of the mix as well as the rate of strength gain.
- A low percentage of 0.5% of active filler was found not adequate to meet the required strength requirements.
- 1.0% of active filler and 2.0% of bitumen content were determined at all the strength and stiffness values as the optimum contents.

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Evaluation of Physical Properties of RAP According to the Rejuvenator

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Recycling Asphalt Concrete
Rejuvenator

ABSTRACT:
Over 90% of road pavement in Korea is made of asphalt concrete. In addition to the vehicle load, the pavement is damaged due to environmental factors such as abnormal temperature. In this case, waste asphalt is generated in the process of cutting the asphalt concrete for the repair work. The waste asphalt thus produced is mixed again in the plant. At present, waste asphalt is put to practical use not only in domestic but also in USA and Europe. Recycling of such waste asphalt is an indispensable element not only in environmental issues but also in domestic limited resources. In addition, the use of asphalt waste is mandatory in conducting asphalt construction according to the guidelines of the Ministry of Land, Infrastructure, and Transport. As a result, the use of recycled asphalt in the construction of asphalt concrete is increasing. It is also economical in terms of cost reduction compared to general asphalt concrete pavement. It can replace natural aggregate by reusing the aggregate of waste asphalt, and it is possible to replace hot aggregate asphalt mixture. It is possible to mix at low temperature, and it is anticipated that environment-friendly effect such as carbon reduction can be achieved. Recycled asphalt basically has different performance depending on recycled additives. The purpose of this study is to evaluate the basic properties of recycled asphalt using three types of mid-temperature regeneration additives used in Korea and to commercialize recycled asphalt which satisfies the quality criteria of domestic asphalt mixture according to test results. In this study, Marshall stability, indirect tensile strength, moisture sensitivity, dynamic stability and exfoliation resistance were evaluated with three samples.
Evaluation of physical properties of RAP according to the Rejuvenator

1. Introduction

In the 21st century, harmony with the environment and conservation of the global environment are becoming more important. Accordingly, recycling of resources in various fields prevents resource depletion and maintains the global environment. Research on the existing construction sector has focused on high performance, but it is focusing on environmentally friendly factors considering the environmental load according to the changing national policy. This situation is also applied to domestic roads. 80% of the construction waste generated annually is waste concrete and waste asphalt. In the case of waste asphalt, asphalt which is less than expected life and destroyed due to increase in number of cars, overloaded truck, extreme environmental factors, etc. are generated and waste asphalt is steadily increasing. In the early days, aggregate generated from waste asphalt was evaluated as uneconomical and insecure, but in the case of recycled aggregate, the quality of natural aggregate was improved to similar level. In case of Kuwait, the feasibility of applying asphalt on waste concrete using waste concrete was studied. As a result, asphalt mixture using recycled aggregate was evaluated by Marshall stability test, moisture sensitivity test, strength loss test and repeated running test. The result was satisfied with the asphalt mixture specification (Ahmad H. Aljassar et al, 2005). A study on the added value of asphalt mixtures using recycled aggregate was conducted in Singapore. The values of the Marshall properties of the mixture using recycled aggregates satisfied the criteria of the Transportation Authority of Singapore. As a result, it is reported that it is possible to replace the recycled aggregate with a part of the asphalt mixture, but long-term performance evaluation is required (Yiik Diew Wong et al., 2006). In February 2009, the Green Growth Committee was launched in Korea. With the vision of entering the world's top five green power nations by 2020, this committee will implement three strategies: climate change adaptation, energy self-reliance, creation of new growth engines, improvement of quality of life. It was established as an organization directly under the presidency in order to promote the nation's green growth. Currently, the Ministry of Land, Transport and Maritime Affairs has been notified of the use of recycled aggregate and recycled aggregate recycled products and the use of compulsory used construction materials such as recycled aggregate. The use of such waste asphalt can conserve limited resources in Korea and allow for economic construction. In addition, this study developed a mid-temperature additive to utilize recycled asphalt at 30 °C lower than that of conventional asphalt. Lowering the heating temperature by 30 °C has a positive impact on global warming, which is a recent issue, as it reduces carbon emissions. So, The performance evaluation of newly developed warm regeneration additive was carried out. Such medium temperature asphalt is being evaluated in many advanced countries and is being commercialized.

2. Material

The asphalt mixture was made by mixing 70% of new aggregate composed of 13mm aggregate and crushed fine aggregate, 3.78% of new asphalt binder of PG64-22 grade widely used in Korea, 30% of recycled asphalt and 1.8% of recycled additive. The asphalt binder aging proceeds with time in the asphalt pavement, and the strength of the aged binder is greatly increased but the ductility is decreased. To solve these problems, recycled asphalt is mixed with recycled additive. Recycled additive recovers ductility and viscosity of aged binder. Two additives (Sample B and C) which are used in domestic market and a newly developed Sample C were made in Korea. The asphalt mixture used in this experiment was applied to the domestic WC-2 mixture standard and was in conformity with the Marshall mix design. Figure 1 is Aggregate gradation.
3. EXPERIMENTAL METHODS

3.1 Production of asphalt concrete specimens.

1) Mixing of materials

Aggregates and asphalt are weighed using a bowl and heated in an oven at the mixing temperature (130 °C for the quality of asphalt). The aggregate is heated until it has been heated to the mixing temperature and then the aggregate and asphalt are mixed. Mixing is carried out within the mixing temperature range so that the asphalt binder can be evenly applied to the aggregate surface.

2) Specification of the specimen

Ensure that the temperature of the asphalt mixture (115 °C, based on the quality of the warm mix asphalt) is within the compaction temperature range. The number of compaction was carried out by compaction of 75 times on both sides. After the two-sided compaction, remove the bottom plate of the mold, cure it for 24 hours so that deformation does not occur, and demold the specimen. After demolding, store on a flat surface for testing.

3.2 Marshall stability test

The Marshall stability test is a test method widely used in Korea for the purpose of mixing design and quality control of heated asphalt mixture. It is a test to measure the maximum resistance and deformation value of a specimen that can resist the plastic flow at a temperature of 60 °C.
After measuring the height of the specimen, the specimens were immersed in a 60 °C water bath for 30 minutes and then the load was measured at a rate of 50.0 mm / min. The time required to take the specimen out of the water tank and to finish the measurement should be within 30 seconds.

3.3 Indirect tensile strength and Toughness test

The indirect tensile strength test is a test to evaluate the resistance of the asphalt pavement layer to fatigue cracks and cold cracks in areas with severe traffic loads or temperature changes. It was carried out according to KS F 2382 regulation in Korea. This is done by placing a compressive load acting on the asphalt mixture specimen in parallel along the radial plane direction perpendicular to the horizontal and vertical strains. A tester with a curvature surface equal to the radius of the Marshall specimen with a diameter of 101.6 mm is used to provide a nearly uniform stress distribution. At this time, load is applied at a speed of 50 mm / min. Toughness is recorded with the indirect tensile strength test measurements. Toughness is a value that evaluates the crack resistance of a mixture by measuring the resistance to deformation that occurs when the load is loaded on the specimen, that is, the extent to which the generated strain energy can be absorbed (Ahn et al. 2016). The Ministry of Land, Infrastructure and Transport of Korea of "Guidelines for the Production and Construction of Asphalt Mixtures" prescribe indirect tensile strengths of 0.8 (N / mm) or more and toughness of 8,000 (N · mm) or more (The Ministry of Land, Infrastructure and Transport of Korea, 2017).

3.4 Moisture Susceptibility test(TSR test).

The asphalt separation is caused by the adhesion problem between the asphalt binder and the aggregate, and the moisture is one of the causes of the asphalt separation. For this reason, several studies have been conducted to investigate the effect of moisture on asphalt, but it is difficult to achieve high reliability. In this study, the Tensile Strength Ratio(TSR) test of domestic KS F 2398 was applied as a moisture Susceptibility test. The moisture Susceptibility test method can be summarized as follows.

- To comply with the target porosity (7 ± 0.5%), compaction is carried out by applying different compaction times to produce three specimens. After compaction, mold is demolded in mold after storing at room temperature for 24 hours. The porosity is calculated by using the theoretical maximum density and the apparent density value of the prepared specimen.
- Classify the specimens satisfying the target porosity into at least three specimens in two categories (dry and moisture treated).
- The specimens to be tested in the moisture-free condition shall be left at room temperature until the test. The specimens to be subjected to moisture treatment shall be immersed in vinyl, such as zipper bags, and subjected to indirect tensile strength test after soaking in a constant temperature water bath at 25 °C for 120 ± 10 minutes.
- Prepare the specimens to be tested in the wetted state after the vacuum treatment in the following order.

1️⃣ Use a coarse aggregate washed from the bottom of the vacuum system to insert a specimen with a clearance between the bottom of the vacuum system and the specimen
② Fill distilled water at least 25 mm from the specimen surface, apply vacuum pressure in the range of 13 ~ 67kPa for 5 ~ 10 minutes and remove vacuum pressure. No vibrations are applied to the bowl.

③ Soak the partially saturated specimens for 5 ~ 10 minutes and freeze them for 16 hours at -18 ± 3 °C.

④ The mixture is soaked in a constant temperature water bath at 60 ± 1 °C for 24 ± 1 hour.

⑤ Take out the specimen from the constant temperature water tank at 60 ± 1 °C and put it in the constant temperature water bath at 25 ± 0.5 °C for 120 ± 10 minutes. Then, the untreated specimen and the water treatment specimen are taken out from the constant temperature water bath at 25 ± 0.5 °C according to KSF 2382 After performing the indirect tensile strength test, calculate the tensile strength ratio.

3.5 Dynamic stability (Wheel-Tracking).

Asphalt reacts differently at high and low temperatures and reacts sensitively to temperature. It has rigidity at low temperature but plastic deformation behavior at high temperature like summer. At high temperatures, the binder reacts flexibly and permanent deformation occurs when a traffic load is applied in that state. In this study, the dynamic stability of the asphalt mixture was determined by using wheel-tracking equipment to evaluate plastic deformation resistance. Dynamic stability is the number of wheels required to sink 1 mm between 45 and 60 minutes when the wheel-tracking tire is loaded. To prepare the specimens required for this test, a mixture of 300 mm * 300 mm * 50 mm was made in the laboratory and the porosity was adjusted to about 4 ± 0.5%. The prepared specimens were subjected to wheel-tracking test at 60 °C after curing for 24 hours.

3.6 Dynamic Immersion Test.

The purpose of the dynamic immersion test is to evaluate the adhesion of the aggregate and the asphalt binder. Prepare the aggregate as dry and clean with a size of 8.0mm ~ 11.2mm, add a certain amount of asphalt and mix. The asphalt mixture is cured at room temperature for 15 hours, and the asphalt mixture and the glass rod are placed in a dynamic immersion glass bottle. The glass bottles were spinning for 24 hours, and the asphalt mixture and the glass rod collide during the rotation of the glass bottle to peel off the asphalt binder. After 24 hours, Degree of covering of asphalt binder is evaluated. At this time, the evaluator must be two or more. This test is included in British Standard as The European Standard EN 12697-11 and is named "Determination of the
affinity between aggregate and bitumen”. Figure 2 shows the reference for estimation of degree of bitumen coverage.

**Guidance for estimation of the degree of bitumen coverage**

Systematic evaluation of the degree of bitumen coverage on aggregate particles may be facilitated by reference to the figures shown below.

![Figure 2 reference for estimation of degree of bitumen coverage.](image)

4. Results


Through the Marshall stability test, the maximum resistance and the maximum strain value that the specimen can withstand the plastic flow can be known. In order to make the temperature sensitive asphalt to plastic state, the specimens were soaked in a 60 °C water bath for 30 minutes and
loaded at a rate of 50.0 mm / min to measure the Marshall stability of Samples. Figure 3 is a Marshall stability result. The Marshall stability quality standard set forth in the "Guidelines for the Production and Construction of Asphalt Mixtures" by the Ministry of Land, Infrastructure, and Transport is stipulated as above 7,500N for 75 on both sides of the Marshall compaction. As shown in Fig. 8, the Sample A and Sample C has a similar value of 13,349N and 13,397N, and the Sample B has a value of 12,590N, which is about 1000 lower than that of the two additives. Therefore, it is considered that Sample A and the Sample C have better plastic flow resistance than the Sample B.

![Figure 3. Marshall stability result](image)

4.2 Indirect tensile strength and Toughness test

Indirect tensile strength and toughness were measured to evaluate the crack resistance of the Sample A and the Sample B and C. The toughness measured in the indirect tensile strength measurement is a value for evaluating the crack resistance of the mixture by measuring the resistance to deformation occurring when the load is loaded on the specimen, that is, the degree of absorbing the generated strain energy. Figure 4 and Figure 5 show the indirect tensile strength and toughness results for samples of Sample A and s. Indirect tensile strength and toughness were measured for both indirect tensile strengths of 0.8 N / mm or more and toughness of 8,000 N / mm or more as defined in the Ministry of Land, Infrastructure, and Transport "Asphalt Mixture Production and Construction Guidelines". Experimental results showed that all of the measured
specimens satisfied the criteria of The Ministry of Land, Infrastructure, and Transport, and that the Sample A had higher indirect tensile strength and toughness than the samples of the other twos.

![Figure 4. Indirect tensile strength test result](image)

![Figure 5. Toughness test result](image)

4.3 Moisture Susceptibility test (TSR test).

The specimens prepared by Marshall compaction were cooled at -18 °C for 16 hours, then indirectly tensile strength was measured by immersed for 24 hours at 60 °C for 2 hours at 25 °C
and indirectly with dry specimens. Water resistance (TSR) was evaluated. The TSR reference value provided in the Ministry of Land, Infrastructure, and Transport guidelines for the production and construction of recycled asphalt mixtures is stipulated as 0.75 or more. Figure 6 shows that the specimens of the Sample A and the Sample B satisfied the moisture resistance standard values as a result of the Moisture Susceptibility test (TSR) test. However, in the case of the specimens of the Sample C, it was confirmed that the specimens had a water resistance value of 0.30, so that it was judged that the water resistance was not sufficiently secured.

![Figure 6 Moisture Susceptibility test result](image)

4.4 Dynamic stability (Wheel-Tracking).

In this study, dynamic stability was checked to evaluate plastic deformation resistance of recycled asphalt prepared with Sample A, B, and C. According to the Ministry of Land, Infrastructure, and Transport Guidelines, the dynamic stability of recycled asphalt mixtures is defined as 750 times or more. Table 1 shows the results of dynamic stability measurement of Samples, except for the Sample B, the specimens using Sample A and the Sample C satisfied the dynamic stability standard value, And the dynamic stability of the specimen using Sample A was 1,048 times higher than that of the specimens of other Samples.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Dynamic Stability</th>
</tr>
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<tbody>
<tr>
<td>A</td>
<td>0.844</td>
</tr>
<tr>
<td>B</td>
<td>0.83</td>
</tr>
<tr>
<td>C</td>
<td>0.304</td>
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</table>

Table 1 Result of dynamic stability
<table>
<thead>
<tr>
<th>Sample</th>
<th>Dynamic stability(Time)</th>
<th>strain(mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1048</td>
<td>6.06</td>
</tr>
<tr>
<td>B</td>
<td>712</td>
<td>10.97</td>
</tr>
<tr>
<td>C</td>
<td>936</td>
<td>7.26</td>
</tr>
</tbody>
</table>

4.5 Dynamic Immersion Test.

Dynamic immersion test was carried out to evaluate the separation resistance of recycled asphalt mixtures using recycled additive. The specimens dynamically immersed in accordance with the BS standard were subjected to a visual inspection by three evaluators. In comparison with Sample A and A, 60% of the Sample A and 50% of the Sample A and the Sample C showed a 40% exfoliation resistance. It is considered that the Sample B has a slightly higher peel resistance than the Sample C specimen.
5. Conclusion

In this study, the performance evaluation of the asphalt mixture of three Samples was carried out to evaluate the performance of moderate regenerating additive. For each asphalt mixture with mid-temperature regeneration additive, evaluation was made as to whether it meets the standards of the asphalt mixture proposed in the "Standards of asphalt mixture production and construction guidelines" of the Ministry of Land, Infrastructure, and Transport.

1. The Marshall test results show that the Marshall stability of the asphalt mixture is higher than the reference value of 7,500N and the Sample A and C have the equivalent Marshall stability of 13,349N and 13,397N respectively, . Sample B has lower Marshall stability of 12,590N. It is considered that Sample A and C have better plastic flow resistance.

2. The indirect tensile strength of Sample A was 1.27 N / mm, which was higher than the reference value of 0.8 N / mm and had a slightly higher indirect tensile strength than the Sample B and C with indirect tensile strength of 1.14 and 1.13 N / mm . The toughness was also 13,468 N / mm for Sample A and 11,349 N / mm and 12,622 N / mm for the Sample B and X, respectively. Sample A values were higher than the others, Indicating that Sample A exhibits excellent crack resistance.

3. As a result of measuring the tensile strength ratio (TSR) of the asphalt mixture, the Sample A and the Sample B were 0.84 and 0.83, respectively, satisfying the TSR reference value of 0.75. This indicates that the Sample A regeneration additive has sufficient moisture resistance.

4. Dynamic stability measurement results showed that the values of Sample A and B and C were 1048.3 and 715.5 and 936, respectively. The results were obtained satisfying the
criterion of 750 or more except for the Sample B. The rutting value was also found to be 6.06mm for the Sample A and 10.97mm and 7.26mm for the Sample B and C, respectively. Therefore, Sample A is more resistant to plastic deformation than others.

5. As a result of evaluation of separation resistance through dynamic immersion, Sample A and B showed a peel resistance of 60%, indicating that the reference value exceeded 50%, but that of Sample C was 40%. It is judged that the separation resistance of the recycled asphalt mixture using Sample A is sufficiently secured.

Acknowledgement

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Reference

Baron Colbert, Zhanping You(2011) The determination of mechanical performance of laboratory produced hot mix asphalt mixtures using controlled RAP and virgin aggregate size fractions, Department of Civil and Environmental Engineering, Michigan Technological University, 1400 Townsend Drive, Houghton, MI 49931-1295, United States
Feasibility Assessment of Using Reclaimed Asphalt Pavement (RAP) in Highway Construction in Bangladesh

Track: Innovative Materials and Technology

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KEYWORDS: RAP, recycling, stabilized base, cement treated base, life analysis

ABSTRACT:

Reclaimed Asphalt Pavement (RAP) is not yet popular in Bangladesh though it depends mostly on imported materials for road construction. This study attempts to investigate the recycling prospects of RAP as a granular base, stabilized base and cement treated base. RAP is collected from the Joydebpur-Mymensingh Road Improvement Project area from the roadside stockpile. Series of laboratory tests are performed on RAP to determine the bitumen content and physical properties of aggregates including aggregate gradation, Los-Angeles abrasion value, and Aggregate Crushing Value (ACV). Specimens are mixed at 160°C and tested for Marshall Stability and flow by adding 0.5%, 1%, 1.5%, 2% of virgin bitumen with the aged materials. Maximum stability of 10 kN is found for the specimen when 1.5% virgin bitumen is added. Modified Proctor test and CBR test are performed to investigate the feasibility of RAP as a granular base and cement treated base. Soaked CBR value for virgin aggregate mixed with 75% RAP and treated with 5% Portland cement is found to be 57%. The study also predicts the life of the flexible pavement with recycled RAP and compares with a new construction with all virgin materials. Prediction of life is estimated by Mechanistic-Empirical (M-E) method where the KENPAVE software calculates critical strains at pavement layers. From the M-E approach, it is found that a stabilized base constructed by RAP materials can replace the binder course and thus reduce the thickness of the pavement as well as the cost of reconstruction.
Feasibility Assessment of Using of Reclaimed Asphalt Pavement (RAP) in Highway Construction in Bangladesh

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1 INTRODUCTION

In Bangladesh, among the various modes of transport, road transport system has been playing a significant role in passengers and freight movement. The road network of Bangladesh consists of national highways, regional highways, and district roads. The total length of roads under the supervision of the Roads and Highways Department is nearly 21,000 km (Musa and Azam 2016).

Most of the roads of Bangladesh are paved with hot mix asphalt (HMA) on the surface. Factors such as aging of HMA layers, rapid growth in traffic volume, and high axle loads necessitate the maintenance and rehabilitation of existing roads, as well as the construction of new roads. According to HDM-4 (Highway Development and Management) analysis, 17,047 km of flexible pavements under RHD needs to be maintained by structural overlay, partial and full reconstruction to keep those in serviceable condition requiring 21,148 2.40 million BDT (Bangladeshi Taka, Approximately BDT 80.0 = US $1.0) for the years from 2016-17 to 2020-21 (Musa and Azam 2016). Generally, in Bangladesh, the national highway pavements consist of the bituminous layer over stone chips base layer and the bottom layer is compacted subgrade (Asaduzzaman et al. 2014). The brick chips aggregate in the subbase is the most popular since it is locally available and cheap compared to crushed stones (Rasel et al. 2011). For maintenance or rehabilitation purpose bituminous overlay is the most commonly used over the existing bituminous layer or the brick chips base layer. Cost of per square meter overlay with a thickness of 40 mm to 59 mm in Bangladesh is 23 thousand USD which is 9% higher than that of India (Collier et al. 2016). One of the causes for the excessive cost for overlay construction is the cost of the construction materials for which it depends mostly on import.

Although Bangladesh has a very limited or no source of construction materials, it does not practice the recycling of the Reclaimed Asphalt Pavement (RAP). However, a review of researches shows that RAP of various percentages has been used in the construction of bituminous pavement varying from 10 to 70% in many countries of the world (Singh et al. 2015). In Japan, the average RAP use in asphalt mixture is 47% whereas in the USA it is slightly more than 20% (West and Copeland 2015). The Colorado Department of Transportation allows up to 25% of RAP to be used as mixes (Copeland 2011). Percentage of RAP used varies from project to project and condition of RAP materials. The reduction in the cost of the project is directly proportional to the percentage of RAP used through an ideal percentage should be analyzed which gives the optimal strength parameters and economics. Preparing a bituminous mix with 100% use of RAP is not advisable. If processed and used appropriately RAP can provide standards either equal or superior to all virgin HMA. Using RAP makes the project more economical & reduces the cost of the project marginally (Singh et al. 2015).

Though recycling of RAP is practiced in many developing countries, it is yet to be demonstrated as a standard method of maintenance or rehabilitation in Bangladesh. In this study, the feasibility of RAP as a stabilized base, granular base and cement treated base has been assessed by laboratory experiments and analysis through KENPAVE software.

2 METHODOLOGY

The research work is performed in three steps including 1) collection of RAP, 2) laboratory investigation of RAP and 3) assessment of RAP to be used as a stabilized base, granular base and cement treated base. The flowchart of whole research work is shown in Figure1.
3 MATERIALS

3.1 Collection of the RAP Materials

The sample was collected from Joydebpur-Mymensingh road improvement project (JMRIP) a part of Dhaka-Mymensingh Highway (N-3). Materials were obtained in April 2017 from the roadside stockpile. According to the Project office, cracks developed in wearing course of the pavement by one year of pavement construction. The damaged wearing course was removed by using an excavator and stockpiled near the road by the project. The RAP remained in the roadside stockpile for six months until it was collected by the study team. According to the Project office, the penetration grade of the bitumen during construction was 60/70. The RAP has gone through further weathering effect as the materials were stockpiled after an extended period. In Figure 2 stockpiling of materials is shown.

![Figure 2: RAP materials stockpiled along the side of the road (N-3)](image)

After collecting the sample sieve analysis, binder extraction test, aggregate crushing value (ACV) test, Los-Angeles abrasion test have been performed to know the physical properties and quality of RAP.
3.2 Binder Contents and Recovered RAP Aggregates

Binder content in RAP was determined by ignition method according to ASTM D6307. Aggregates were recovered after burning the RAP in asphalt binder analyzer machine at 5500 °C for 2 hours to remove the asphalt coating. The recovered aggregates were mostly angular, and the nominal maximum aggregate size was 19 mm. Sieve analysis was performed to check whether it meets the RHD requirement. The particle size distribution of aggregates after removal of binder from the RAP is shown in Figure 3. From the graph, it is found that the particle size distribution curve for test material lies below the lower limit curve of RHD for wearing course (RHD 2011). Figure 4 shows the photo of RAP before and after binder extraction test.

![Figure 3: Particle distribution of RAP aggregate after removing the binder](image)

Figure 3: Particle distribution of RAP aggregate after removing the binder

![Figure 4: Photos of RAP before and after binder extraction test](image)

Figure 4: Photos of RAP before and after binder extraction test

3.3 Milled RAP Aggregates

RAP had been milled with stone crusher machine to get the desired gradation for aggregate base layer required by (RHD 2011). In Bangladesh, appropriate milling machine to reclaim RAP from the pavement is not readily available as reuse/recycle of RAP is not yet practiced by the road agencies. Figure 5 shows the particle size distribution graph of the milled RAP. Since a standard milling machine was not used to collect the RAP aggregates, the gradation in Figure 5 is different from that of the Figure 3 which represents the actual aggregates used during the construction.
4 EXPERIMENTAL ASSESSMENT OF RAP TO JUSTIFY DIFFERENT ALTERNATIVE SOLUTIONS

4.1 RAP is to be Used as Stabilized Base

Stabilized base is a base layer where load carrying capacity is increased by the use of cement or asphalt with aggregates, and consequently, a thinner wearing course can be used over the stabilized base. Tests were conducted to identify the Marshall stability and flow values of the specimens with an aim to use the RAP as a stabilized base.

Marshall Stability and flow tests were conducted on the specimens until desired stability was found. Four tests were performed with the addition of virgin bitumen from zero to two percent with an increment of 0.5% of the total weight of the RAP specimens which already had some aged binder. The Penetration grade of the virgin bitumen used for this study is 80/100 which was collected from Eastern Refinery Ltd, Bangladesh, in a sealed container. The mixture was compacted using 75 blows on each side of the mold by a Marshall compactor. Here, mixing temperature was set to be 160 °C, and compaction was completed before it downfall to 50 °C. 100% RAP was used for the tests.

4.2 RAP is to be Used as Granular Base

In a granular base, no cementing material is used, and aggregates are interlocked together due to frictional resistance generated for the surface to surface contact. Water helped the material to get bonded through proper compaction. No heat energy is required in this process. The process is comprised of three stages 1) crushing RAP by a stone crusher machine, 2) finding the optimum moisture content by modified Proctor test conforming to ASTM D 1557 and 3) conducting CBR test in soaked condition according to ASTM D 1883.

4.3 RAP is to be Used as Cement Treated Base

CBR test had been performed on one specimen composed of 75% RAP, 25% virgin aggregate, and 5% ordinary Portland cement (OPC). Aggregates were blended as well as cement had been added at the presence of moisture of optimum quantity to increase the CBR value that was found in the granular base. One Soaked CBR test was performed with this specimen. 4% optimum moisture determined from the modified Proctor test was used as the mix moisture. The total curing time of the mold was seven (7) days. Figure 6 shows some of the photos of laboratory tests.
Figure 6: Pictures of Marshall Flow and stability test and CBR test

5 PAVEMENT LIFE ANALYSES BY KENPAVE

The KENPAVE analysis is performed to understand the design life of partially reconstructed roads with RAP. Design life comparisons are made between a reconstructed road with virgin layers and that of using RAP.

5.1 Mechanistic-Empirical (M-E) method

The mechanistic-empirical method of design is based on the mechanics of materials that relate inputs, such as wheel load, pavement layer thicknesses, layer modulus, and Poisson’s ratio of layers to an output or pavement response, such as stress or strain. The strain values of critical locations are used to predict the life of the pavement. Dependence on observed performance is necessary because theory alone has not proven sufficient to design pavements realistically (Huang 2004). Using KENPAVE software, Horizontal tensile strain ($e_t$) at the bottom of the surface layer and Vertical compressive strain ($e_c$) at the top of the Subgrade were found out. Then using two empirical equations fatigue life ($N_f$) and rutting life ($N_d$) were calculated.

5.2 Fatigue Criterion

The fatigue equation employed by the Asphalt Institute for a standard mix with an asphalt volume of 11% and an air void volume of 5% is

$$N_f = 0.0796(e_t)^{3.291} \times (E)^{0.851}$$

(1)

where, $N_f$ is the allowable number of load repetitions to control fatigue cracking, and $E$ is the dynamic modulus of the asphalt mixture, and $e_t$ is the horizontal tensile strain at the bottom of the surface course (Huang 2004).
5.3 Permanent Deformation Criterion

The allowable number of load repetitions to control permanent deformation can be expressed as

\[ N_d = 1.365 \times 10^{-9}(e_c^{4.477}) \]  

(2)

As long as proper compaction of the pavement components is obtained and the asphalt mix is well designed. Here, \( N_d \) = allowable number of load repetitions to control permanent deformation, \( e_c \) = Vertical compressive strain at the top of the subgrade (Huang 2004).

6. RESULTS AND DISCUSSIONS

Table 1 shows the results of the physical properties of aggregates. It is found that ACV value and LA value are within the limit for requirements of the aggregate base of RHD. According to Table 2, Figure 7 & 8, at 4% bitumen content [aged bitumen (2.5%) plus virgin bitumen (1.5%)] maximum stability was found. It satisfied the RHD and Asphalt Institute specifications (Asphalt Institute 2014). However, minimum flow value for this bitumen content is higher than the allowable limit. At 3% bitumen content [aged bitumen 2.5% plus 0.5% virgin bitumen] both the stability and flow value are within the allowable limit. So, stability value at 3% bitumen content is considered for analysis for pavement life.

Table 1: Physical properties of RAP

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<tr>
<th>Name of the test</th>
<th>Test Result</th>
<th>RHD specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sieve Analysis</td>
<td>Bitumen content= 2.5%</td>
<td>Gradation curve</td>
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<tr>
<td>Nominal max aggregate size</td>
<td>19 mm</td>
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<td>Binder Extraction Test</td>
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<tr>
<td></td>
<td></td>
<td>Base type-II &lt; 35</td>
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<tr>
<td>ACV Test</td>
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<tr>
<td>Los Angeles Abrasion test (LAA)</td>
<td>35%</td>
<td>Base type-I ≤ 35</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Base type-II ≤ 40</td>
</tr>
</tbody>
</table>

Table 2: Marshall stability and flow values for specimens with different virgin bitumen

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>Added Virgin Bitumen (%)</th>
<th>Stability (KN)</th>
<th>Flow (0.25mm)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>5.2</td>
<td>9.2</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>0.5</td>
<td>8</td>
<td>14</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>8.5</td>
<td>16.4</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>1.5</td>
<td>10</td>
<td>16.8</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>2</td>
<td>9</td>
<td>16.8</td>
<td></td>
</tr>
</tbody>
</table>

Bitumen content in RAP was 2.5%
According to Figure 9, CBR value for 100% granular RAP is significantly lower than that with cement treated RAP with 25% virgin aggregate. For 100% RAP CBR value was found 12.5% which is too low to be used as a base course. As the crushed materials were not densely graded and no virgin aggregate was used very low CBR value was obtained. For 100% RAP, CBR value remains close to 20% as found in different research papers (Saha & Mandal 2017). On the other hand, specimen of 75% RAP, 25% Virgin aggregates and 5% cement mixed at optimum moisture content (4%) shows a CBR value of 57%. Due to the blending of finer particles with the crushed RAP, good CBR value was achieved. So, it can be used as in the base layer (RHD 2011).

Different layer properties of Joydebpur-Mymensingh Highway used for KENPAVE analysis are shown in Table 3

<table>
<thead>
<tr>
<th>Layer</th>
<th>Thickness (cm)</th>
<th>Elastic Modulus (MPa)</th>
<th>Poisson’s Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bituminous Overlay</td>
<td>5</td>
<td>3200</td>
<td>0.30</td>
</tr>
<tr>
<td>Bituminous Course</td>
<td>17</td>
<td>3200</td>
<td>0.30</td>
</tr>
<tr>
<td>Base Type I</td>
<td>15</td>
<td>200</td>
<td>0.35</td>
</tr>
<tr>
<td>Base Type II</td>
<td>23</td>
<td>150</td>
<td>0.35</td>
</tr>
<tr>
<td>Sub-Base</td>
<td>35</td>
<td>100</td>
<td>0.35</td>
</tr>
<tr>
<td>Improved Subgrade</td>
<td>30</td>
<td>80</td>
<td>0.40</td>
</tr>
<tr>
<td>Subgrade</td>
<td>-</td>
<td>50</td>
<td>0.40</td>
</tr>
</tbody>
</table>
The elastic modulus was used 2758 MPa by using conversion chart from Marshall stability to elastic modulus when stabilized RAP replaced bituminous binder course (Fwa T.F. 2006). Similarly, elastic modulus was used 175.820 MPa by using conversion chart for CBR value to elastic modulus when base Type-I was replaced (Fwa T.F. 2006). The analysis is done for different combinations of recycled RAP based pavement using these layer properties. The graphical representations for fatigue and rutting life for different combinations and their comparisons are shown in Figure 10 & 11.

Figure 10: Comparison of fatigue life for different combinations of RAP based reconstruction

From the comparison it can be seen that while using RAP as a stabilized base with an 8cm overlay gives us the maximum fatigue life which is more than the life of full new reconstruction. While using an as granular base with cement treatment almost gives us the same value as complete new reconstruction.

Figure 11: Comparison of rutting life for different combinations of RAP based reconstruction

From the comparison it can be seen that while using RAP as a stabilized base with an 8cm overlay gives us the maximum rutting life which is way more the life of full new reconstruction. While using an as granular base with cement treatment almost gives us the same value as complete new reconstruction. So results are the same whether it is fatigue life or rutting life.
7 CONCLUSIONS

Recycled Reclaimed Asphalt Pavement can be a feasible alternative to virgin materials for pavement reconstruction. In Bangladesh where pavement reconstruction works are done frequently, RAP can perform as a useful element of pavement reconstruction regarding cost effectiveness, sustainable environment and availability. The findings of the study are summarized as below:

(1) Marshall Stability and flow values were found 8 kN and 14 accordingly at 3% bitumen content (aged bitumen 2.5% plus virgin bitumen 0.5%). From the analysis, it was also observed that RAP as stabilized base with 8 cm overlay had given us the fatigue life and rutting life more than full new reconstruction. So, according to the results RAP can be used as a stabilized base.

(2) CBR value of 100% granular RAP was found 12.5% which did not fulfill the criteria of Roads & Highway technical specifications (RHD 2011). So, the study finds that granular base of 100% RAP is not technically feasible.

(3) CBR value of the test specimen of 75% RAP, 25% virgin aggregate and 5% Portland cement has observed 57% CBR value which satisfied the RHD specifications for the base. It was also noted from the analysis that this combination had given almost same fatigue and rutting life to full new reconstruction.

Every year many roads are reconstructed in Bangladesh which generates lots of RAP, and most of them are kept by the side of the road or get wasted. So, if RAP can be reused through recycling, these available materials can be useful and the need for virgin or new aggregates will be reduced. Not only it will lessen the cost of reconstruction but also will be good for the environment.

8 ACKNOWLEDGMENTS

The authors acknowledge the excellent co-operation and support of all the officials and laboratory staffs of the Department of Civil Engineering, MIST, Dhaka, and Special Works Organization (SWO-West). The authors are also great full to Roads & Highway Officials (RHD) for their kind help during this research work.

9 REFERENCES

RHD 2011. Technical Specifications, Bangladesh
<table>
<thead>
<tr>
<th>PAPER TITLE</th>
<th>Development of the Multipurpose Chemical Additive for Warm Mix Asphalt</th>
</tr>
</thead>
<tbody>
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<td>TRACK</td>
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</tr>
<tr>
<td>AUTHOR</td>
<td>(Capitalize Family Name)</td>
</tr>
<tr>
<td>Katsura ENDO</td>
<td>Chief Researcher</td>
</tr>
<tr>
<td>CO-AUTHOR(S)</td>
<td>(Capitalize Family Name)</td>
</tr>
<tr>
<td>Tadashi TSUNEMATSU</td>
<td>Senior Researcher</td>
</tr>
<tr>
<td>Akane IKEDA</td>
<td>Researcher</td>
</tr>
<tr>
<td>E-MAIL</td>
<td>(for correspondence)</td>
</tr>
</tbody>
</table>

**KEYWORDS:**
Warm mix asphalt, Multipurpose chemical additive, RAP

**ABSTRACT:**
A multipurpose chemical additive is newly developed for Warm Mix Asphalt. This new additive enables producing and applying at a temperature around 30 to 50 degrees in Celsius lower than an equivalent Hot Mix Asphalt. General dosage is around 0.5 to 1.0% by weight of asphalt, i.e. only 250 to 500g by weight of 1,000kg mix in case of 5% of asphalt content. Low dosage means cost effective even supposing that the unit price of the additive is a little high. This additive is effective for not only straight-run asphalt, but also any polymer modified asphalt and specific decoloring asphalt which are commercially available in Japan. And this additive is effective for any graded mixtures and for both virgin and RAP used mixtures. Furthermore, the thermal oxidative aging using this additive is evaluated in two ways. First, dense-graded mixtures with straight-run asphalt and this additive are compacted after 72-hour heated at 160oC. Second, dense-graded mixtures with 50% of RAP and this additive are compacted after 24-hour silo storage without anti-oxidant. Few changes in mixture properties are found in both cases. These effects are verified through many paving projects in Japan. For example, porous asphalt pavement with polymer modified asphalt is well paved under -1 degrees in Celsius in daily average temperature with light snow. These pavements are still in good condition compared to adjacent general pavements.
I INTRODUCTION

Conserving the global environment has been one of the major problems for a long time. Act on promotion of procurement of Eco-friendly goods and services by the State and other entities (Green Purchasing Law) has been in effect since May 31, 2000 in Japan and business operators and citizens in Japan are to endeavor to select Eco-friendly goods to the extent possible when purchasing or leasing goods and/or receiving the provision of services. The designated procurement items list which includes Eco-friendly goods is documented and items on the list are promoted to be used.

Paving works are same. For example, hot mix asphalt (HMA) with reclaimed asphalt pavement (RAP) has been on the list since March 2004 and warm mix asphalt (WMA) without RAP has been on the list since February 2010. WMA technologies has been studied since early 1990s in Japan and some commercialized technologies were developed. Most were oil-based technologies and few were chemical foaming technologies.

The Nippon Road had also two technologies for WMA but these had some disadvantages comparing to other products. The first is that these depend on asphalt, i.e. the oil-based technology is only for straight-run (non-modified) asphalt and the chemical foaming technology is only for modified asphalt. The second is that dosage depends on volume of newly adding asphalt in case of WMA with RAP. And the third is that total binder tends to increase in case of HMA and/or WMA with RAP because rejuvenator is required.

The authors have been developed a new multipurpose chemical additive for the use of WMA and overcame above disadvantages. This paper describes the outline of the additive, some examples of mix properties with this additive, some case studies and so forth.

2 BACKGROUND OF NEEDS FOR A MULTIPURPOSE ADDITIVE

2.1 Trends of HMA Production in Japan

Publish investment has been decreased and decreased since Japan's overheated stock and real estate
markets had also been collapsed in early 1990s. Concerns about improvement of infrastructure including road buildings and pavement repairs declined at the same time. Figure 1 shows the amount of HMA production from 1984 to 2016 (Japan Asphalt Mixture Association 2018). Maximum total amount was about 80 million tons in 1992 and it falls to about one-half in 2016 according to the latest statistics.

On the other hand, HMA with RAP has been gradually increased ever since then. As a heavy black line in this figure shows, percentage of HMA with RAP was about 15% in 1992, it reached to 50% in 1998 and is about 77% in 2016.

![Figure 1. Amount of HMA Production in Japan (Japan Asphalt Mixture Association 2018)](image)

As noted above, WMA without RAP was on the designated procurement items list in 2000. Virgin HMA was no longer main product at that time. WMA without RAP is one of the products of virgin HMA. It was said that percentage of WMA was under 1% of all HMA products.

2.2 Trends of Polymer Modified Asphalt in Japan

Japan has currently more than 1.2 million km road network. As shown in Figure 2, maximum number of large vehicles was about 21 million in 1991. Since then, severe rutting has been one of the major sources of concern for not only road authorities but also road users. Using polymer modified asphalt (PMA) is one of the solutions for that. PMA type I includes rubber as the modifier and PMA type II includes a kind of resin as the modifier. Both have become a general product in 1988. Since then totally seven PMA have been developed and used in Japan according to their purpose and application, as shown in Table 1. The amount of production of HMA using PMA was about 380,000 tons. It is about 11% of all HMA. And the amount was 480,000 tons and about 16% of all HMA, according to the latest statistics (Japan Modified Asphalt Association 2015). PMA usually used for virgin HMA or WMA. As already shown in Figure 1, about 23% of total production is virgin
HMA and this means that PMA is used for about 70% (=16/23) of virgin HMA.

From these, any additives for the use of HMA or WMA using virgin asphalt only was not needed. And development goal should be one additive for the use of HMA or WMA using PMA and HMA or WMA with RAP.

![Figure 2. Number of Vehicles in Japan (Automobile Inspection & Registration Information Association 2017)](image)

**Table 1. Types of PMA and Its Application in Japan (Japan Modified Asphalt Association 2006)**

<table>
<thead>
<tr>
<th>Purpose</th>
<th>Application</th>
<th>Type I</th>
<th>Type II</th>
<th>Type III</th>
<th>Type III-W</th>
<th>Type III-WF</th>
<th>Type H</th>
<th>Type H-F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent deformation</td>
<td>General road</td>
<td>Very good</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Heavy duty road</td>
<td>Very good</td>
<td></td>
<td></td>
<td>Very good</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Heavy duty road and crossing</td>
<td>Very good</td>
<td>Good</td>
<td>Good</td>
<td>Good</td>
<td>Good</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wearing</td>
<td>Cold and snowy area</td>
<td>Very good</td>
<td>Very good</td>
<td>Good</td>
<td>Good</td>
<td>Good</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Water sensitivity</td>
<td>Concrete deck bridge</td>
<td>Good</td>
<td>Good</td>
<td>Very good</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flexibility</td>
<td>Steel deck bridge</td>
<td>Good</td>
<td>Good</td>
<td>Very good</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Permeability</td>
<td>Surface</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Very good</td>
<td>Very good</td>
</tr>
<tr>
<td>Available mixture</td>
<td>Dense-graded, fine-graded, coarse-graded mixtures</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Porous asphalt mixture</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

2.3 Study about Another Option

The authors understand that foaming technologies for WMA are mainly used in the United States (NAPA 2007). The Nippon Road imported one of the foaming technologies from USA about eight years ago.
and made some tries to produce WMA with the foaming process. Finally, the Nippon Road decided that the foaming technology is not applicable as WMA in Japan because it was hard to decrease producing and paving temperature about 30 degree in Celsius comparing to generally produced HMA and asphalt pavement laid using this technology ‘moved’ slightly after opened to traffic. This phenomenon sometimes led to rutting in early stage.

3 BRIEF DESCRIPTION OF DEVELOPED CHEMICAL ADDITIVE

Figure 3 shows developed chemical additive. It is white and solid and flake form at ambient temperature and melting point is 53 degree in Celsius. This additive is easy to dissolve in any asphalt, so it is not necessary to extend mixing time when producing WMA. This additive has high-temperature stability, then it is applicable for silo storage and long-distance hauling of produced mixtures. pH is 7.1, i.e. this is neutral. No need to care about environmental affairs due to acidity and alkalinity, if this additive leaks from pavement after service.

![Figure 3. Developed Chemical Additive](image)

Actual dosage depends upon mixtures and asphalt contents, and general dosage is 0.5% to 1.0% by weight of asphalt. So only 250g to 500g addition is required to produce 1,000 kg of WMA with this additive when asphalt volume is 5.0%.

Figure 5 shows effects to asphalt penetration according to dosage of this additive and Figure 6 shows effects to asphalt softening point. In these figures, 0.0% of dosage mean the original values of asphalt used. Penetrations are slightly increase with dosage of additive. But it seems that the impacts are limited when dosage is under 1.0%. Softening points show almost same values of the original asphalts. Thus this additive is not a modifier of asphalt.
Figure 4. Effect to Penetration of Asphalt

Figure 5. Effect to Softening Point of Asphalt

4 CASE STUDIES

4.1 Mix Design of Dense-graded Asphalt Mixture

The Marshall mixture design method is still adopted in Japan. Figure 6 shows the comparison of air voids of cylindrical specimen. Selected aggregates measured to meet dense-graded mixture requirement. PMA type II is used in this case with a purpose of higher deformation resistance. Recommended mixing temperature of this PMA is 170 to 185 degree in Celsius and recommended compaction temperature is 160 to 170 degree in Celsius. Some trial mix are mixed at 178 degrees in Celsius and compacted at 168 degrees in Celsius. According to the Marshall method, OAC is 4.9% and shows 3.6% air voids. No additive is used in this stage. Specification of air voids for dense-graded mixture is 3 to 6%. This mixture meets all specifications including air voids. This is referred as ‘general HMA’ in Figure 6.

Other specimens are made with the new chemical additive. This additive is used after introducing asphalt during mixing. Dosage is 0.8% by weight of asphalt, i.e. only 0.5g for about 1,200g mixture. To confirm the effect of this additive both mixing and compaction temperatures have been changed as showed in Table 2.
Specification of air voids of dense-graded mixture is 3 to 6% in Japan. Air void of general HMA is 3.6%. On the other hand, air voids of WMA with this additive are 2.7%, 2.7%, 2.9% and 3.3% when compaction temperatures are 170, 150, 130 and 110 degree in Celsius respectively. This result indicate that this additive can improve the compactability of asphalt mixture and enables compaction temperature drop more than 50 degree in Celsius relative to general HMA.

![Figure 6. Comparison of Air Voids of Cylindrical Specimen](image)

Table 2. Mixing and Compaction Temperatures when new additive is used

<table>
<thead>
<tr>
<th>Case</th>
<th>Mixing Temperature (degree in Celsius)</th>
<th>Compaction Temperature (degree in Celsius)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>175 (-3)</td>
<td>170 (+7)</td>
</tr>
<tr>
<td>Case 2</td>
<td>155 (-23)</td>
<td>150 (-13)</td>
</tr>
<tr>
<td>Case 3</td>
<td>135 (-43)</td>
<td>130 (-33)</td>
</tr>
<tr>
<td>Case 4</td>
<td>115 (-63)</td>
<td>110 (-53)</td>
</tr>
<tr>
<td>General HMA</td>
<td>178</td>
<td>163</td>
</tr>
</tbody>
</table>

Note: Number in a parenthesis means temperature difference relative to general HMA

4.2 Field Test Pavement

The authors conducted small test pavements to confirm the temperature drop efficiency of this additive at a private field in Tsukuba, Japan. Dense-graded mixture, coarse-graded mixture and porous asphalt mixture were constructed. Maximum aggregate sizes are 13mm, 20mm and 13mm, respectively. Test section had 2.8m in width and 90m in length. Thickness was 5cm. Table 3 shows mix design results for each mixture. The new additive is not used in mix design stage. After some trial mix, dosages of the additive are decided as 0.7%, 0.5% and 0.7% by weight of asphalt, respectively. And target temperature drop of mixing are 30 degree in Celsius in every case.
Table 3. Mix Design Result

<table>
<thead>
<tr>
<th></th>
<th>Dense-Graded</th>
<th>Coarse-Graded</th>
<th>Porous Asphalt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt used</td>
<td>Pen. 60-80</td>
<td>Pen. 60-80</td>
<td>PMA Type H</td>
</tr>
<tr>
<td>Asphalt Content (%)</td>
<td>5.5</td>
<td>4.7</td>
<td>4.8</td>
</tr>
<tr>
<td>Air Void (%)</td>
<td>3.9</td>
<td>3.9</td>
<td>20.5</td>
</tr>
<tr>
<td>Density (g/cm³)</td>
<td>2.387</td>
<td>2.416</td>
<td>1.980</td>
</tr>
<tr>
<td>Stability (kN)</td>
<td>10.9</td>
<td>10.6</td>
<td>6.4</td>
</tr>
<tr>
<td>Mixing Temperature (degree in Celsius)</td>
<td>152</td>
<td>152</td>
<td>170</td>
</tr>
</tbody>
</table>

All mixtures were produced at a batch plant. This test filed was at adjacent to the asphalt plant, so paving works were carried out after one hour hauling to simulate general paving works at public roads. MF43WD asphalt finisher made by Mitsubishi was used to spread the produced asphalt mixtures. The SW650 tandem roller with 7 tons made by Sakai was used for the first compaction and the TX701 pneumatic-tired roller with 15 tons made by Sakai was used for the finishing compaction. A steel-tired roller is generally used for the first compaction in Japan, however it was not able to use due to the narrow filed width. Table 4 shows a series of measured temperatures of asphalt mixtures at each construction stage. Mixture production temperatures and spreading temperatures are almost on target and/or below target.

Table 4. Measured Temperatures of Asphalt Mixtures at Each Construction Stage

<table>
<thead>
<tr>
<th></th>
<th>Dense-Graded Mixture</th>
<th>Coarse-graded Mixture</th>
<th>Porous Mixture</th>
<th>Asphalt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Target Production Temperature</td>
<td>122 (-30)</td>
<td>122 (-30)</td>
<td>140 (-30)</td>
<td></td>
</tr>
<tr>
<td>Production Temperature</td>
<td>122 (-30)</td>
<td>119 (-33)</td>
<td>140 (-30)</td>
<td></td>
</tr>
<tr>
<td>Mixture Temperature After Hauling</td>
<td>123</td>
<td>123</td>
<td>140</td>
<td></td>
</tr>
<tr>
<td>Spreading Temperature</td>
<td>101 (-39)</td>
<td>100 (-40)</td>
<td>98 (-57)</td>
<td></td>
</tr>
<tr>
<td>First Compaction Temperature</td>
<td>80</td>
<td>78</td>
<td>88</td>
<td></td>
</tr>
<tr>
<td>Finishing Compaction Temperature</td>
<td>52</td>
<td>54</td>
<td>75</td>
<td></td>
</tr>
</tbody>
</table>

Note: Unit is degree in Celsius. Number in a parenthesis means temperature difference relative to general HMA

Six cores with 100 mm in diameter are sampled at each mixture site and measured their densities and air voids. And percent of lab-compacted densities are calculated. These results are shown in Table 5. Percent of lab-compacted density means density of WMA core divided by lab-compacted HMA specimen’s density. These results show three WMA are well compacted even though production temperature is dropped 30 degree relative to general HMA.

Table 5. Density and air voids of Sampled 100mm Cores

<table>
<thead>
<tr>
<th></th>
<th>Dense-Graded Mixture</th>
<th>Coarse-graded Mixture</th>
<th>Porous Mixture</th>
<th>Asphalt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (g/cm³)</td>
<td>2.421</td>
<td>2.450</td>
<td>2.051</td>
<td></td>
</tr>
<tr>
<td>Air Voids (%)</td>
<td>4.2</td>
<td>4.0</td>
<td>19.4</td>
<td></td>
</tr>
<tr>
<td>Percent of lab-compacted density (%)</td>
<td>98.1</td>
<td>98.2</td>
<td>99.0</td>
<td></td>
</tr>
</tbody>
</table>

4.3 Construction Work at a National Road

The Nippon Road made a contract on a national road in Tokyo. This was a construction for wire
common conduct. The wire common conduct was buried under a road in service and pavement should be restored daily to open traffic. This work was carried out at night. So actual construction hours were very limited. Taking consideration of shortening the construction hours, it sometimes faces higher temperature of HMA. The Nippon Road decided to adopt WMA. And as previously noted, this WMA should be included RAP due to Green Purchasing Law.

As a result of mix design, WMA with 50% RAP was adopted for asphalt stabilized base material for the base course, coarse-graded mixture for binder course and dense-graded mixture for surface course. Table 6 shows the mixture properties and conditions. Every mixture is WMA with RAP, but no rejuvenator is used because this new additive also acts like a rejuvenator (Endo et.al. 2016).

<table>
<thead>
<tr>
<th>Table 6. Asphalt Mixtures Properties and Conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Property</strong></td>
</tr>
<tr>
<td>Asphalt Content (%)</td>
</tr>
<tr>
<td>Air Void (%)</td>
</tr>
<tr>
<td>Lab-compacted Density (g/cm³)</td>
</tr>
<tr>
<td>Production Temperature (°C)</td>
</tr>
<tr>
<td>Compaction Temperature (°C)</td>
</tr>
<tr>
<td>Penetration of Adding Asphalt (1/10mm)</td>
</tr>
<tr>
<td>Additive Content (%)</td>
</tr>
<tr>
<td>RAP Content (%)</td>
</tr>
<tr>
<td>Rejuvenator</td>
</tr>
</tbody>
</table>

This was two-year long project. General HMA with RAP was also in this work. Figure 7and Figure 8 show temperature drop comparison between HMA with RAP and WMA with RAP after placing binder course and surface course. HMA data were collected during winter season and average air temperature was 3.1°C. And WMA data were collected during fall season and average air temperature was 11.2°C. Red dotted line shows a target temperature to enable opening for traffic. 20 minutes reduction for binder course construction and 15 minutes reduction for surface course construction in case of WMA with RAP comparing to HMA with RAP. Taking account for the air temperature, it is expected that WMA with RAP has more reduction of construction hours effect.

Table 7 shows the percentage of lab-compacted density of filed sampled cores of HMA with RAP and WMA with RAP. Specification of percentage is 96% or above. All cases meet this specification and WMA with RAP are well compacted like HMA with RAP even though WMA with RAP were produced and constructed about 30℃ temperature decrease comparing to HMA. HMA with RAP property was not noted in this paper, rejuvenator was used. These facts show that developed new additive act well as not only warm-mix additive but also rejuvenator. Other mixture properties like stability, rutting resistance and so forth were on same levels.
Figure 7. An example of Surface Temperature Drop after Placing Binder Course

Figure 8. An Example of Surface Temperature Drop after Placing Surface Course

Table 7. Percentage of Lab-compact Density of Filed Cores

<table>
<thead>
<tr>
<th></th>
<th>#</th>
<th>Asphalt Stabilized Material</th>
<th>Coarse-graded Mixture</th>
<th>Dense-graded Mixture</th>
</tr>
</thead>
<tbody>
<tr>
<td>WMA with RAP</td>
<td>1</td>
<td>100.4</td>
<td>98.7</td>
<td>96.6</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>101.4</td>
<td>101.5</td>
<td>96.9</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>101.4</td>
<td>99.5</td>
<td>97.6</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>101.2</td>
<td>101.3</td>
<td>98.5</td>
</tr>
<tr>
<td>Ave.</td>
<td></td>
<td>101.1</td>
<td>100.3</td>
<td>97.4</td>
</tr>
<tr>
<td>HMA with RAP</td>
<td>1</td>
<td>98.9</td>
<td>100.3</td>
<td>97.5</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>102.5</td>
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<td>100.6</td>
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</tr>
<tr>
<td>Ave.</td>
<td></td>
<td>100.6</td>
<td>101.1</td>
<td>98.5</td>
</tr>
</tbody>
</table>

4.4 Other Projects

Some examples of other projects are shown in Table 8. This list summarized in 2012 and old data because this additive was developed in 2010 and the authors eagerly collected such data at that time. After that, the authors had few interested in these records because this additive is communized within the Nippon Road. As well-known, WMA has other aspect, i.e. it enables longer distance hauling than HMA and also enables enough compaction of pavement during cold weather. Fiscal year in Japan starts in April and ends in next
March. It is said that this leads to more construction works in winter season. This list includes such cases.

Table 8. Examples of Other Projects (As of 2012)

<table>
<thead>
<tr>
<th>Area</th>
<th>Mixtures</th>
<th>Asphalt</th>
<th>Amount of Production (tons)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Osaka</td>
<td>Coarse-graded, Asphalt Stabilized</td>
<td>Pen.60-80</td>
<td>2,300</td>
</tr>
<tr>
<td>Ibaragi</td>
<td>Coarse-graded, Asphalt Stabilized</td>
<td>Pen.60-80</td>
<td>9,000</td>
</tr>
<tr>
<td>Fukushima</td>
<td>Dense-graded</td>
<td>PMA Type II</td>
<td>500</td>
</tr>
<tr>
<td>Kanagawa</td>
<td>Dense-graded</td>
<td>PMA Type II</td>
<td>2,000</td>
</tr>
<tr>
<td>Hiroshima</td>
<td>SMA</td>
<td>PMA Type II</td>
<td>200</td>
</tr>
<tr>
<td>Fukushima</td>
<td>SMA</td>
<td>PMA Type II</td>
<td>300</td>
</tr>
<tr>
<td>Kouchi</td>
<td>Porous asphalt pavement</td>
<td>PMA Type IH</td>
<td>100</td>
</tr>
<tr>
<td>Hokkaido</td>
<td>Porous asphalt pavement</td>
<td>PMA Type IH</td>
<td>780</td>
</tr>
<tr>
<td>Fukushima</td>
<td>Coarse-graded, Asphalt Stabilized</td>
<td>Pen.60-80, PMA Type II</td>
<td>11,000</td>
</tr>
<tr>
<td>Tokushima</td>
<td>Coarse-graded, Asphalt Stabilized</td>
<td>Pen.60-80</td>
<td>300</td>
</tr>
<tr>
<td>Kouchi</td>
<td>SMA</td>
<td>De-colored Asphalt</td>
<td>200</td>
</tr>
<tr>
<td>Iwate</td>
<td>Dense-graded with RAP</td>
<td>Pen. 60-80</td>
<td>430</td>
</tr>
<tr>
<td>Other</td>
<td>Dense-graded with RAP</td>
<td>Pen. 60-80</td>
<td>4,000</td>
</tr>
</tbody>
</table>

5 CONCLUSIONS
This paper concludes like followings.
- A new chemical additive for WMA was developed.
- This additive does not affect to asphalt properties.
- This additive can use for any asphalts including polymer modified asphalt.
- This additive can use for any asphalt mixtures.
- General dosage of this additive is only 0.4 – 0.8% by weight of asphalt.
- This additive enables 30 – 50 ℃ temperature decrease comparing to general HMA.
- This additive acts like rejuvenator when this is used for WMA and/or HMA with RAP.
- Some case studied are shown to understand above features.

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**PAPER TITLE**
Development of a Master Function for Calculation of the Interlayer Bond Shear Stiffness and its Impact on the Service Life of Asphalt Pavements

<table>
<thead>
<tr>
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<tr>
<th>AUTHOR (Capitalize Family Name)</th>
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<th>ORGANIZATION</th>
<th>COUNTRY</th>
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<tbody>
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<td>Professor</td>
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</tr>
</tbody>
</table>

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**KEYWORDS:**
Interlayer bond, finite element method, shear stiffness, master function, fatigue

**ABSTRACT:**
A good and durable interlayer bond is crucial for a long service life of asphalt pavements. In order to determine the shear stiffness at the interface between asphalt layers and to take into account the interactions of repeated traffic loading, acceleration and braking processes as well as temperature influence, a complicated apparatus for cycling testing of the interlayer bond has been developed. An extensive experimental procedure has been created to include all factors that influence the interlayer bond. Using the experimental results, a master function for the analytical assessment of the interlayer bond shear stiffness has been established. Through implementation of the master function into a finite element program the fatigue status of asphalt pavements, which is affected by the interlayer bond of different quality, have been calculated over the service life of 30 years using the German method for computational design according to RDO Asphalt 09. Using these fatigue functions, it has been succeeded to make recommendations for the practical use for both maintenance and new asphalt pavement construction.
1 INTRODUCTION

A good and durable interlayer bond is crucial for a long service life of asphalt pavements. In order to determine the shear stiffness at the interface between asphalt layers and to take into account the interactions of repeated traffic loading, acceleration and braking processes as well as temperature influence, a complicated apparatus for cycling testing of the interlayer bond has been developed. An extensive experimental procedure has been created to include all factors that influence the interlayer bond. Using the experimental results, a master function for the analytical assessment of the interlayer bond shear stiffness has been established. Through implementation of the master function into a finite element program the fatigue status of asphalt pavements, which is affected by the interlayer bond of different quality, have been calculated over the service life of 30 years using the German method for computational design according to RDO Asphalt 09. Using these fatigue functions, it has been succeeded to make recommendations for the practical use for both maintenance and new asphalt pavement construction.

The results presented below are based on the results of IGF project No. 17634 BG "Cyclic Shear Stiffness and Shear Fatigue Testing for Evaluation and Optimization of Interlayer Bond in Asphalt Pavements", supported by the Association of Industrial Research Communities (AIF) of the German Asphalt Institute (DAI) in cooperation with the Institute for Road Research of the TU Braunschweig.

1.1 BACKGROUND

The construction of asphalt concrete pavements usually consists of a surface course, a binder course and a base course. Asphalt base courses are usually produced in two layers when the thicknesses is more than 17 cm. The layers are bonded together through a tack coat (e.g. bitumen emulsion). The asphalt concrete pavement is loaded permanently in both a vertical direction by vehicle’s wheel loads and a horizontal direction by braking and acceleration processes. Additionally, temperature variation also cause additional stresses. The interlayer bond (IB) should be produced as a full-surface and rigid connection between the individual layers of the asphalt pavement in order to allow the transmission of shear stresses between them. The effect of this bond has to be generated through the interlocking of the aggregate particles at the interface, the friction between the surfaces of the two asphalt layers and the adhesion between the asphalt binder of the two layers and the applied tack coat. When transferring shear stresses across the layer interface, these three factors act simultaneously in different proportions, depending on the asphalt mixes, the temperature, the normal pressure, and the type and quantity of the bitumen emulsion.

Missing or too flexible interlayer bond changes the three-dimensional stress state in the entire pavement structure, which results in a reduced service life due to premature material fatigue. In order to ensure that all layers of the asphalt pavement act as a unit during the load transfer and that no or only very small relative displacements occur at the layer interface between the layers, the production of a best possible rigid interlayer bond must be striven.

1.2 OBJECTIVES

The main objective of this study is to develop a master function for analytical assessment of the shear stiffness, which takes into account the dependence between shear stiffness, temperature, shearing frequency and normal stress in order to implement the established master function in the finite element program SAFEM (Semi-Analytical FE Method). In conjunction with the German method for computational design RDO Asphalt 09 it is aimed to computationally estimate the effect of different shear stiffness functions for qualitatively different interlayer bonds over the service life of the whole asphalt pavement.
2 EXPERIMENTAL PROGRAM

2.1 DOUBLE-LAYERED ASPHALT SPECIMENS AND APPARATUS FOR CYCLIC TESTING OF THE INTERLAYER BOND

Two-layered asphalt slabs (320 mm x 260 mm) were prepared in the roller sector compactor using a compression program with position-controlled pre-compression and force-controlled main compression. The slabs of the underlying course were produced and stored at room temperature (RT) for 24 hours. The bitumen emulsion was then applied uniformly using a flexible foam roller. The coated slabs were left at RT for at least two hours until the complete breaking of the bitumen emulsion (AL Sp-Asphalt 09). The hot bituminous mixture of the upper course was then laid and compacted. Four cylindrical specimens (Ø100 mm) were drilled from one double-layered asphalt slab (Figure 1, left). The asphalt specimen was fixed inside two steel adapters with the aid of a two-component epoxy adhesive. The gap between the two steel adapters was set to 1.0 mm and the interface of the specimen was precisely adjusted to fit in this gap (Figure 1, middle).

![Figure 1. Sample preparation for CTIB.](image)

The test apparatus used for the tests was designed to apply cyclic shear force in the vertical direction and static normal force in the horizontal direction and was mounted in the temperature chamber of a servo-hydraulic testing machine (Figure 2, left). The test sample was inserted and fixed in the jowls A and B, so that half of the sample was in A and the other half was in B (Figure 2, right). The gap between the jowls was the same as it was between the steel adapters of the sample (1.0 mm). The steel adapters were fastened in the jowls with screws to avoid any possible movement of the sample in the test device. The sinusoidal shear cyclic loading was applied to one layer of the specimen (jowl B) by the hydraulic cylinder of the servo-hydraulic testing machine and was position-controlled. The second half of the specimen was held unmovable in vertical direction by jowl A. The normal pressure was applied on the back of the asphalt specimen (jowl A) by a piston rod through a steel plate. The vertical shear displacement of jowl B and the horizontal motion of jowl A was measured by four sensors.
2 EXPERIMENTAL PROCEDURE

The test procedure started always at a temperature of $T = -10^\circ\text{C}$, normal stress $\sigma_N = 0.9$ MPa, shearing frequency $f = 10$ Hz and a maximal shear displacement $s_{w,max} = 0.03$ mm. For each specimen it ended at $T = 50^\circ\text{C}$, $\sigma_N = 0.9$ MPa, $f = 10$ Hz and $s_{w,max} = 0.15$ mm. The whole experiment was conducted at four different temperatures. At each temperature the specimen was loaded with five normal stresses. Six frequencies at the corresponding number of load cycles changed successively during each normal pressure. The whole procedure of simultaneous and consecutive process runs was fully automated and no manual interference was required. The duration of the whole procedure for one specimen lasted 11h 43min.

2 FUNCTIONS FOR THE ASSESSMENT OF SHEAR STIFFNESS

2.1 DETERMINATION OF THE MASTER FUNCTION

The basis for determining the master function from the test results was the following procedure:

$$\tan \gamma_s = \frac{\tau_s}{G_s} \tag{1}$$

For small shear angles $\tan \gamma_s \approx \gamma_s$. The shear strain can be determined as the ratio of the shear displacement $s_w$ to the gap between the steel adapters $d_s = 1.0$ mm

$$\tan \gamma_s = \gamma_s = \frac{s_w}{d_s} \tag{2}$$

The shear stiffness is therefore

$$G_s = \frac{\tau_s}{\gamma_s} = \frac{\tau_s}{\left(\frac{s_w}{d_s}\right)} \tag{3}$$

with
\[
\tau_s = \frac{F_s}{A}
\] (4)

where \(F_s\) is the shear force amplitude (N), \(\gamma_s\) is the shear strain (-), \(\tau_s\) is the shear stress (MPa), \(A\) is the cross section at the interface (mm\(^2\)), \(s_w\) is the shear displacement amplitude (mm), \(d_s\) is the gap between steel adapters (mm), and \(G_s\) is the shear stiffness (MPa/mm).

The development of a universal master function, which takes into account the combined influence of temperature, normal stress and frequency was necessary for the analytical assessment of the shear stiffness. The regression which approximated the experimental values most accurately was the sigmoid function (Figure 4), which approaches asymptotically the minimum and the maximum values of the shear stiffness. The temperature-frequency equivalence as described in AL Sp-Asphalt 09 was computed as follows:

\[
m = \log(\alpha_T \cdot f) = \log \left( \frac{1}{e^{\frac{E_a}{R} \left( \frac{1}{T+273.15} - \frac{1}{T_R+273.15} \right)}} \cdot f \right)
\] (5)

It was found that the master function for the shear stiffness \(G_s\) was as follows

\[
G_s = G_{s,\text{max}} + \left( G_{s,\text{max}} - G_{s,\text{min}} \right) \frac{1 + e^{m \cdot \log(\alpha_T \cdot f)}}{1 + e^{m \cdot \alpha_T \cdot f}}
\] (6)

where \(m\) is the temperature-frequency equivalence (Hz); \(G_{s,\text{min}}\) is the minimal shear stiffness (MPa/mm); \(G_{s,\text{max}}\) is the maximal shear stiffness (MPa/mm); \(\alpha_T\) is the shift factor (-); \(f\) is the frequency (Hz); \(T\) is the test temperature (°C); \(T_R\) is the reference temperature (20°C); \(E_a\) is the activation energy (J/mol); \(R\) is the universal gas constant (J/mol·K).

![Figure 4. Functional dependence between the shear stiffness and the temperature-frequency equivalence at four normal stresses.](image)

The regression parameters, \(a\) and \(b\), are both functions of the normal stress. For parameter \(a\) the logarithmic function showed the best fit, while for parameter \(b\) it was the linear function.
\[
\begin{align*}
a &= c_1 \cdot \ln \sigma_N + c_2 \\
b &= d_1 \cdot \sigma_N + d_2
\end{align*}
\]  
where \(c_1, c_2, d_1, d_2\) are the function parameters.

After substituting the regression parameters \(a\) and \(b\) in Equation 6, the latter becomes:

\[
G_s = G_{s,\min} + \frac{\left(G_{s,\max} - G_{s,\min}\right)}{1 + e^m(c_1 \cdot \ln \sigma_N + c_2 + d_1 \cdot \sigma_N + d_2)}
\]  
(9)

Using this master function, the shear stiffness of the interlayer bond can be calculated for any arbitrary temperature, frequency and normal stress.

The sigmoidal master functions for calculating the shear stiffness at the two interfaces of road pavements (surface course – binder course and binder course – base course) were then inserted into the Semi-Analytical Finite Elements Method program SAFEM (Oeser et al. 2014), in order to investigate the fatigue of the road pavement influenced by the interlayer bond.

2.2 SHEAR STIFFNESS OF COMPLETELY FATIGUED INTERLAYER BOND

From the experimentally determined fatigue curves at the Technical University of Braunschweig it was found, that the increase in the normal stress causes generally an increase in the remaining shear stiffness over the entire service life of the road pavement as well as a higher number of load cycles until complete fatigue (Figure 5). The fatigue curves approached the same value range at all three temperatures for each normal stress, which leads to the conclusion that the functions of the completely fatigued interlayer bonds are independent of the temperature.

![Figure 5. Fatigue curves for the interlayer bond at two normal stresses and three temperatures.](image)

Under the assumption that the fatigue is normal stress but not temperature dependent, a linear relationship (10) of the shear stiffness and the normal stress between the average values of the completely fatigued interlayer bonds was determined (Figure 6), namely

\[
G_s = 42.82 \cdot \sigma_N + 4.025
\]  
(10)

Using this function, the service life of the completely fatigued interlayer bond was calculated in SAFEM.
3 RESULTS

The extensive experiments have shown that the shear stiffness between two asphalt layers does not exceed a shear stiffness of 100 MPa (Wellner et al. 2016). The interlayer bond is thus significantly weaker than it is considered in the calculation model of RDO Asphalt 09.

In order to investigate the effects of the reduced (flexible) interlayer bond on the fatigue behavior of asphalt pavements, calculations according to the RDO Asphalt 09 method were performed using the finite element program SAFEM. For this purpose, five different interlayer bond configurations were defined for the layer boundaries between surface and binder course, as well as between binder course and base course:

1. Full interlayer bond (FIB):
   - theoretical case,
   - upper and lower asphalt layer are firmly connected,
   - assumed for the dimensioning calculation according to RDO Asphalt between asphalt layers.
2. Good Case (GC): experimentally established best shear stiffness on laboratory-produced test specimens,
3. Bad Case (BC): minimum level of shear stiffness on the laboratory-produced test specimens,
4. Completely fatigued interlayer bond (CFIB): determined in long-term tests normal-stress-dependent, temperature-independent shear stiffness (please see chapter 2.2)
5. Completely missing interlayer bond (CMIB):
   - theoretical case,
   - upper and lower asphalt layer slide smoothly on each other.

The cases listed under 1. and 5. were not proven to exist in the experiments, but they are shown for orientation. Another question was, what impact a bitumen-based adhesion (flexible and non-rigid interlayer bond) has particularly taking into account additional interfaces, e.g. for base layers with a thickness of more than 17 cm.

The results have been produced for a construction according to Table 1, line 1 of the RStO 2012 for a dimensioning-relevant load B corresponding to loading class Bk100. The “Good Case” interlayer bond configuration was set as a reference value for the 100% fatigue status of the asphalt pavement for a service life of 30 years.

For the calculations, following assumptions were made for all possible bonded layers (Figure 7).
3 interfaces:

![Diagram showing three interfaces: Surface course - Binder course, Binder course - Base course, Base course - Base course.]

Figure 7. Possible interlayer bond interfaces.

All calculated fatigue curves are shown in Figures 8 to 11 below.

![Fatigue curves normalized to BC-BC-BC (three interlayer bond interfaces).](image)

Figure 8. All calculated fatigue curves normalized to BC-BC-BC (three interlayer bond interfaces).

For a better overview, a few cases are presented and discussed in the following illustrations. For a better orientation, the cases of rigid and completely missing interlayer bond at all interfaces as well as the reference case BC-BC-BC are always displayed. Figure 9 shows the combination cases BC-BC-BC, BC-BC, GC-GC-GC and GC-GC. A reduction of the service life by 25 … 30% results solely from the consideration of a third interface. Thus, the consideration of three instead of only two interfaces shows the clear effect of the interlayer bond on the service life of the whole asphalt pavement.
Figure 9. Comparison between three instead of two interfaces under otherwise identical conditions.

Figure 10. Comparison between variants Bad Case and Completely Fatigued Interlayer bond.

Figure 10 shows that almost the same service life has been estimated when using the Bad Case bond with three interfaces as it is in the case with two interfaces (surface course on binder course and binder course on base course).
Almost 50% service life reduction was calculated for the completely fatigued interlayer bond at all three interfaces, compared to the reference variant BC-BC-BC.

If different variants with a completely fatigued interlayer bond are assumed at all three interfaces (Figure 11), it becomes clear, that the interfaces between binder course and base course and between base course and base course cause similar service life reductions. The variant with completely fatigued interlayer bond, calculated in both combinations (binder course on base course and base course on base course) hardly shows a change in the estimated service life compared to the variant with a completely fatigued interlayer bond at all three interfaces. The results show, that the interfaces between binder course and base course and between base course and base course are the relevant ones with respect to the fatigue of the asphalt base courses.

**4 POSSIBILITIES FOR MONETARY EVALUATION**

Furthermore, it was investigated, whether a monetary assessment, based on the calculation results, is possible in case that missing bond between the layers after completion of the asphalt pavement is stated. For this assessment, the German "Recommendations for the execution of construction contracts when using RDO Asphalt 09" was used. In the example presented in Figure 12, it should be assessed, what effect the lack of bond between binder and base course can have. The mathematical estimation shows a service life reduction of 5 years or 17%.
Figure 12. Service life reduction in the absence of bonding between binder and base course.

The monetary evaluation can be made using the recommendations for the execution of construction contracts (Figure 13). The red-edged area shown in this figure is enlarged in Figure 14.

Figure 13. Construction cost penalty depending on the service life reduction.

From Figure 14 it can be seen, that a service life reduction of 17% would result in a construction cost penalty of 8%.
Figure 14. Construction cost penalty at a service life reduction of 17%.

Figure 15 shows that in case of a missing bond at both relevant interfaces, a service life reduction of 13 years or 43% is calculated. This situation is shown in Figure 16. Such service life reduction would result in a construction cost penalty of about 33%, which would possibly necessitate the replacement of the defective asphalt pavement.

Figure 15. Service life reduction in the absence of bonding between binder and base course and between base and base course.
5 CONCLUSIONS

A new test apparatus and an extended test procedure have been developed to determine the shear stiffness at the interface for different conditions. A master function for the analytical assessment of the shear stiffness $G_S$ has been established and implemented in the finite element program SAFEM to estimate the influence of the interlayer bond on the service life of asphalt pavements.

Using the finite element calculations and applying the German method for computational design according to RDO Asphalt 09 fatigue curves for a full / rigid (FIB), a good (Good Case), a most achievable (Bad Case), a completely fatigued (CFIB) and a completely missing (CMIB) interlayer bond have been established for an exemplary assumed asphalt pavement. The detailed calculations presented in this paper show that a computational service life assessment on the basis of results from performance testing of both the interlayer bond and the asphalt properties represent plausible and promising possibilities for estimating the quality of new as well as existing asphalt pavements.

6 ACKNOWLEDGEMENTS

The authors would like to thank the German Federation of Industrial Research Associations "Otto von Guericke" (AiF) for financial support as well as the support from the DAL. Furthermore, the authors would like to express their gratitude to the RWTH Aachen Institute for Road Research for providing the finite element program SAFEM for the calculations.

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Estimation for Road Surface Temperature change Pattern using simplified variables

**KEYWORDS:**
Road surface temperature; Sensor; Machine Learning; pattern; MATLAB

**ABSTRACT:**
The road surface condition and temperature is important for road maintenance and safety. To estimate the road surface condition and temperature, the RWIS (Road Weather Information System) is used. However, RWIS is not measured the continuous road surface information but measured the locational road surface information. To overcome the current RWIS limitation, the mobile thermal sensors (thermal mapping sensor) which can collect the road condition employed in some countries. Although the thermal mapping sensor can collect the continuous road surface information, it is difficult to collect big data due to apply few probe car. This study suggests a specific methodology for the prediction of road surface temperature using vehicular ambient temperature sensors and collect road surface and vehicular ambient temperature data on the defined survey route in 2015 and 2016 year, respectively. To find out the correlation between road surface and ambient temperature which may affect patterns of road surface temperature variation, the various weather and topographical conditions along with the test route were considered.

For modelling, all types of collected temperature data should be classified into response and predictor before applying a machine learning tool such as MATLAB. Through data learning using machine learning tool, models were developed and finally compared predicted and actual temperature based on average absolute error. According to comparison results, model enables to estimate actual road surface temperature variation pattern along the roads very well. Model III is slightly better than the rest of models in terms of estimation performance.
Estimation for Road Surface Temperature change Pattern using simplified variables

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1 INTRODUCTION

The number of traffic accidents involving injuries increases in winter due to road surface freezing. A study overseas comparing results of various data types between winter and non-winter seasons also indicated that traffic accidents in winter have increased by about 19% and the number of casualties has also increased, by about 13% as compared to the non-winter seasons (Black and Mote, 2015).

The road surface freezing in winter is thus a grave threat to drivers. Skid-related traffic accidents due to frozen road surfaces are complexly due to exogenous factors such as weather conditions, conditions of snow-removing work, or carelessness of drivers. Nonetheless, the most fundamental reason for skid-related traffic accidents is due to the reduced coefficient of friction between the road surface and the vehicle’s tires, as the road surface temperature is dropped. Thus, if appropriate monitoring of the road surface temperatures is taken, and if road management and traffic operation strategies are established properly, human casualties and property damage due to skid-related accidents in winter can be reduced considerably. A currently available method to determine a road surface temperature can be divided into three parts: First, road surface temperature sensors are installed on the road pavement surface to measure temperature data as similar to a traffic volume detector. Although this method is widely used in general, it can only acquire temperature data of specific region rather than the entire road, and sensors are easily damaged during pavement repair or maintenance. Second, road surface temperatures are measured via infrared method. This method is less expensive than the above-mentioned sensor method. However, it has been known to produce less reliable data. Third, a mobile road surface temperature system (thermal mapping) is attached to vehicles thereby acquiring data on road surface temperature in the road section while driving. This method also utilizes the infrared method. The above-mentioned three methods each have their own advantages and disadvantages. In particular, they have difficulties in measuring road surface temperature in overall road sections, and even if they may be able to measure overall road sections, they are not cost effective. In a previous study, thermal mapping equipment was utilized to collect road surface temperatures and the reliability of the collected data was evaluated through statistical methods. Furthermore, a method of how to divide a road based on evaluated road surface temperature was proposed (Yang et al., 2016).

In the present study, a new approach that can collect, utilize, and predict a large amount of temperature data as well as being cost-effective based on previous studies has been attempted. In other words, the present study developed a model that estimated a pattern of change in road surface temperature by utilizing temperature data collected from external temperature sensors embedded in general vehicles through machine learning.

2 LITERATURE REVIEW

VTT Technical Research Centre of Finland has been developing a black ice detection and warning technology, by which warnings about frozen road conditions are sent to drivers, as shown in Fig. 1 to prevent skid-related traffic accidents due to frozen road surfaces in winter. The road is determined to be frozen based on changes in speeds of traffic and rotating shafts collected from a large number of vehicles and weather information (VTT Technical Research Center of Finland, 2013). In the USA, a prediction model of black ice occurrences was developed to predict regions of black ice occurrences and provide dangerous road section information. Road weather information systems and various meteorological data was collected and utilized as required data (Bukkapatnam et al., 2014). Furthermore, a technology that distinguishes dryness, wetness, and frozen condition in road surfaces is currently commercialized and utilizes imaging equipment, such as stereo cameras and remote sensing technologies (Jonsson, 2011; Omer and Liping, 2010).

Currently, studies on road pavements where an anti-freezing system have been underway in Korea and some road sections have applied the system already. The Korea Expressway Corporation developed a freezing rain prediction system and applied the pilot system to the Yeongdong Expressway. The above-mentioned system developed or is developing technologies utilizing somewhat complex models, and most of them are utilized for the purpose of road management from public sector viewpoints. The method proposed in the present study does not have much cost and can estimate overall road networks since it collects outdoor air temperature data from an unspecified number of vehicles and estimates road surface temperature through machine learning methods. In the road traffic sector, machine learning methods have been applied for a long time to estimate various patterns. Machine learning refers to research on
automated calculation methods of knowledge acquisition processes. Its general, the strategy is to discover a pattern from the learning data group. These patterns are used to predict behaviors of new data. The main categories of machine learning methods are as follows (Mahapatra and Bose, 2001):

- Rule induction
- Neural networks
- Case-based reasoning
- Genetic algorithm
- Inductive logic programming

A study by Antoniou and Koutsopoulos (2006) identified a relationship between traffic speed and density by applying machine learning methods. Their study proved that accuracy of speed estimation can be significantly improved via machine learning methods more than other existing models. A study by Chen and Xie (2015) applied a machine learning technique to identify patterns related to driving behaviors of large commercial freight drivers. As a result, the risk of traffic accidents related to large commercial freight vehicles was able to be predicted. In Korea, Jeon et al. (2016) also applied a machine learning method to analyze a large amount of weather information and traffic condition data to predict traffic flow. The present study proposed a system that predicted weather and related traffic congestion sections using a machine learning algorithm operated in a distributed system environment. Korea Advanced Institute of Science and Technology (KAIST), in Korea, is now studying a method that predicts future traffic situations and shares the results using big data analysis and machine learning techniques. It can predict conditions such as speed, traffic flow, and density, based on infrastructure data, such as video detection system (VDS), dedicated short range communications (DSRC), and toll collection systems (TCS). The developed “REALTraffic” engine for that purpose produced the most optimized prediction results via existing data-based estimations and simulations (IT Donga, 2016).

As described in the above, cases of utilization of machine learning algorithms that have been applied to the road traffic field have increased steadily and have been deemed to be a highly appropriate approach when details cannot be described minutely through simple mathematical models.

3 DATA COLLECTION

The analysis region in the present study was a two-way 34km section from SungDong IC to Isanpo IC, in which outdoor air temperatures and road surface temperature data was collected five times in October 2015 for the first time. The same types of data was collected four times in the same section in October 2016. At the time of collection, ambient temperature and humidity data was collected in addition to outdoor air temperature. Table 1 presents a summary of data and weather conditions collected in 2015 and 2016. A, B, and C in Table 1 refer to sunny and windless conditions, cloudy and a light wind conditions, and cloudy, humid, and windy conditions, respectively.

Table 1. Data Overview

<table>
<thead>
<tr>
<th>Date</th>
<th>Origin</th>
<th>Destination</th>
<th>Weather Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oct 16, 2015</td>
<td>SungDong IC</td>
<td>Isanpo IC</td>
<td>B</td>
</tr>
<tr>
<td>Oct 23, 2015</td>
<td>Isanpo IC</td>
<td>SungDong IC</td>
<td>B</td>
</tr>
<tr>
<td>Oct 23, 2015</td>
<td>SungDong IC</td>
<td>Isanpo IC</td>
<td>B</td>
</tr>
<tr>
<td>Oct 26, 2015</td>
<td>Isanpo IC</td>
<td>SungDong IC</td>
<td>C</td>
</tr>
<tr>
<td>Oct 28, 2015</td>
<td>SungDong IC</td>
<td>Isanpo IC</td>
<td>A</td>
</tr>
<tr>
<td>Oct 12, 2016</td>
<td>Isanpo IC</td>
<td>SungDong IC</td>
<td>B</td>
</tr>
<tr>
<td>Oct 25, 2016</td>
<td>Isanpo IC</td>
<td>SungDong IC</td>
<td>B</td>
</tr>
<tr>
<td>Oct 26, 2016</td>
<td>Isanpo IC</td>
<td>SungDong IC</td>
<td>B</td>
</tr>
<tr>
<td>Oct 26, 2016</td>
<td>Isanpo IC</td>
<td>SungDong IC</td>
<td>C</td>
</tr>
</tbody>
</table>

For data collection, a thermal mapping system was utilized that can collect road surface temperature, ambient temperature, and humidity. This system can be mounted to a vehicle and has an advantage of mobile sensors that can be
installed anywhere. The sensor of the thermal mapping system utilized in the present study satisfies an error range.

Meteorological equipment that is proposed in the method of meteorological instrument was inspected by the Korea Meteorological Institute. For mechanical thermometers, the maximum error range was recommended at ±1.0℃, and for electronic thermometers, the maximum error range was recommended at ±0.3℃. Since thermometers used in the present study were electronic thermometers, a sensor that satisfied an error range of ±0.3℃ was selected. Furthermore, for humidity sensors, an error range of the mechanical type was recommended at ±5% while that of the electronic type was recommended at ±3%. Thus, the present study selected equipment for ambient humidity measurements whose error range was within ±3% to satisfy the allowable range. In the inspection vehicle, a global positioning system (GPS) device that determined a data measurement position and computer that collected, displayed, and stored data including various software programs were mounted. As for outdoor air temperature sensors mounted on the vehicle, two sensors were installed in each of the front and rear parts of the inspection vehicle. The sensor transferred information of currently measured temperatures and humidity through the universal serial bus (USB) connection terminal in real time.

In 2015, 4,665 records, and in 2016, 3,187 records of road surface temperature data was collected respectively, and half of them were randomly sampled for analysis and used in model construction through machine learning. The other half of them were used to evaluate the constructed model.

4 ANALYSIS METHODOLOGY

As for outdoor the core of machine learning is representation and generalization. Representation refers to assessment of data, and the generalization refers to processing yet unknown data. The basic principle of machine learning is to train data to find a relationship between predictor and response variables iteratively thereby estimating a response value with regard to a new predictor.

In Yang et al. (2016), similarity in pattern of changes in road surface temperature was verified under the same route and same weather conditions within a limited environment. They applied a well-known statistical method called correlation analysis. The analysis result proved that a clear statistical significance was found in patterns of changes in road surface temperature at the given conditions. In their study, the most important variable in relation to prediction of changes in road surface temperature was outdoor air temperature collected from two vehicle sensors (front and rear outdoor air temperature sensors) attached to the inspection vehicle. Thus, the main predictor considered during modeling utilized these two sets of outdoor air temperatures, and parts of humidity and ambient temperature data collected at the thermal mapping system were additionally utilized to construct a model.

4.1 Comparison between road surface temperature and outdoor air temperature in 2015

Half of the road surface temperatures collected in 2015 were randomly sampled. This was because the performance of the constructed model through machine learning was thought to be limited due to limited temperature data, as the inspected road section and environment were somewhat limited. The data was used for training to construct Model I. The response variable in Model I was a road surface temperature, and front and rear outdoor air temperature data was used as the predictor. Fig. 2 shows an example of the comparison between road surface temperature and front and rear outdoor air temperature data. The vertical line in the figure represents a boundary of the data collection dates.

Fig. 2 (A) Rear Ambient Temp, (B) Front Ambient Temp and Road Surface Temp. for Development of Model I
The present study calculated an average absolute error (AAE) to measure a similarity between two time-series data, which is shown in the upper right in Fig. 2. The AAE can be calculated as follows:

\[
\text{AAE} = \frac{1}{n} \sum_{i=1}^{n} |\text{time series 1}_i - \text{time series 2}_i|
\]

Where time series 1\(_i\) refers to the \(i\)\(^{th}\) data piece in time series 1 and time series 2\(_i\) refers to the \(i\)\(^{th}\) data piece in time series 2.

The AAE of rear outdoor air temperature in the inspection vehicle was 1.6\(^{\circ}\)C, whereas that of front outdoor air temperature was 1.0\(^{\circ}\)C, indicating the front outdoor air temperature was closer to the actual road surface temperature. Overall, front outdoor air temperature tended to be higher than the corresponding road surface temperature. This was because the front outdoor air temperature sensor was mounted in front of the vehicle, which was vulnerable to be affected by the internal engine temperature and heat from the vehicle. In contrast, a rear outdoor air temperature tended to be lower than actual road surface temperature. Although the road surface temperature in the inspected section showed stable conditions without significant changes overall, a sudden change in temperature within a short period of time was also revealed, as presented in inspection data on Oct. 26 and 28, in which the outdoor air temperature sensor did not respond to the change with the needed sensitivity.

4.2 Estimation result of road surface temperature based on data of 2015

The other randomly sampled data was employed to evaluate Model I. Fig. 3 shows the comparison between road surface temperatures and data used to evaluate Model I.

Fig. 3 also shows a similar result of AAE with that of Fig. 2, as well as differences in temperature observed on Oct. 26 and 28. Thus, it was concluded that the similarity between road surface temperature and front and rear outdoor air temperatures used in model construction and assessment were not significantly different. Fig. 4 shows the comparison results between the estimate of pattern of changes in road surface temperature using Model I and actual pattern of changes in road surface temperature.
When comparing patterns of estimated changes in road surface temperature and actual changes in road surface temperature, its AAE was 0.25°C, indicating that the estimated road surface temperature was closer to the actual road surface temperature than the front and rear outdoor air temperature on average. In addition, the estimated results also responded to sudden changes in temperature on Oct. 26 and 28 with sensitivity.

Model I was constructed using 2,333 records, which was half of the all collected road surface temperature data, and the other half were utilized to evaluate the model to verify the accuracy of Model I by comparing the estimated pattern of changes in road surface temperature and the actual pattern of changes in road surface temperature. If a pattern of change in road surface temperature was estimated simply using collected outdoor air temperature without model application, AAE was expected to range from 1.0°C to 1.7°C. In contrast, if the constructed model was used, AAE was around 0.25°C. In summary, improved similarity can be obtained by 75%–85% if the constructed model is used as compared to that of using outdoor air temperature data to estimate a change in road surface temperature.

4.3 Comparison between road surface temperature and outdoor air temperature in 2016

Two models were developed and evaluated based on data collected in 2016. As done in 2015, random sampling was conducted based on road surface temperature data collected in 2016, and half were extracted to construct the models. In Model II, outdoor air temperature, humidity, and ambient temperature were taken into consideration, whereas they were not considered in Model III. That is, Model II was constructed to analyze the effect of humidity and ambient temperature with regard to pattern estimates of changes in road surface temperature. Fig. 5 shows the comparison between road surface temperature and front and rear outdoor air temperature. Fig. 6 shows an example of humidity and ambient temperature data additionally collected in 2016.

Fig. 5 (A) Ambient Temp. of Rear, (B) Ambient Temp. of front and Road Surface Temp. for Development of Model II

In terms of similarity between outdoor air temperature and road surface temperature, data of 2016 was not significantly different from data of 2015. The only difference was that the AAE of front outdoor air temperature was increased by 0.1°C compared to that of 2015.

Fig. 6 (A) Humidity, (B) Ambient and Road Surface Temp. for Development of Model III
Humidity and ambient temperature showed constant values without significant changes during the collection period. Note that although ambient temperature was lower than road surface temperature, it was sensitive to daily changes in temperature. Thus, it was expected to be useful to estimate a pattern of changes in road surface temperature. Model II was constructed using all road surface temperature data in Figs. 4 and 5 while Model III was constructed using only data in Fig. 5.

4.4 Estimation result of road surface temperature based on data of 2016

As done in 2015, the estimated pattern of change in road surface temperature through Models II and III and the pattern of change in the actual road surface temperature were compared and evaluated, based on the other half of the randomly sampled data. The data used in the evaluation is shown in Figs. 7 and 8.

![Fig. 7 (A) Ambient Temp. of Rear, (B) Ambient Temp. of front and Road Surface Temp. for Evaluation of Model II](image1)

![Fig. 8 (A) Humidity, (B) Ambient and Road Surface Temp. for Evaluation of Model II](image2)

A pattern of change in road surface temperature was estimated after front and rear outdoor air temperatures in the inspection vehicle (Fig. 6) and humidity and ambient temperatures (Fig. 7) were inputted to Model II. The comparison results with actual change in road surface temperature are shown in Fig. 8.

![Fig. 9 Estimated Road Surface Temperature using Model II](image3)
In Fig. 8, the actual change in road surface temperature on Oct 16 was around ±2.5°C, which seemed to be larger than that of other dates in the same year. The estimated change in road surface temperature was also expected to have a similar pattern. However, outdoor air temperatures also had similar problems as shown in Fig. 6. The AAE of the estimated road surface temperature using the model was 0.29°C, which was better than estimation of road surface temperature using outdoor air temperature in terms of comparison. Fig. 9 shows the estimated results of road surface temperature using only outdoor air temperatures without humidity and ambient temperatures.

![Fig. 10 Estimated Road Surface Temperature using Model III](image)

4.5 Analysis on integrated data

The performance of models constructed through machine learning depends on the amount and quality (appropriate correlation between responses and predictors) of trained data. Thus, in this section, Model IV was constructed by using only front and rear outdoor air temperature data of the inspection vehicle, which were common data collected in 2015 and 2016, allowing the model to be evaluated. Fig. 11 shows front and rear outdoor air temperatures and road surface temperatures used to construct Model IV. Fig. 12 shows front and rear outdoor air temperatures and road surface temperatures used to evaluate Model IV.

![Fig. 11 (A) Ambient Temp. of Rear, (B) Ambient Temp. of front and Road Surface Temp. for Development of Model IV](image)

![Fig. 12 (A) Ambient Temp. of Rear, (B) Ambient Temp. of front and Road Surface Temp. for Evaluation of Model IV](image)
Fig. 12 shows the comparison and analysis results between the patterns of change in road surface temperature estimated using front and rear outdoor air temperatures in Fig. 11, using Model IV, constructed based on data in Fig. 10 and the actual pattern of change in road surface temperature. Overall, the figure shows no significant difference with that of the integrated analysis results of 2015 and 2016. However, the similarity based on AAE revealed that the AAE of Model IV was larger than that of the other three models. The comparison and analysis results between the patterns of changes in road surface temperatures in 2015 and 2016 showed that most road surface temperatures were distributed in a range of 15°C–20°C. Furthermore, patterns of changes in corresponding outdoor air temperatures showed that although rear outdoor air temperatures in 2015 were 2°C–3°C lower than road surface temperatures, and they followed the pattern of changes in road surface temperature well. However, a different pattern of changes in outdoor air temperatures was exhibited from a pattern of change in road surface temperature in 2016, and a difference from road surface temperature varied depending on collection date. Specifically, the data integration results of 2015 and 2016 showed that as the absolute amount of training data was increased heterogeneity between training data used to estimate a specific range of road surface temperature (15°C–20°C) was significantly large, which degraded the estimation performance. Conclusively, although an amount of training data is important to guarantee a certain estimation performance from a given model, the quality of data that can describe within the temperature spectrum is also critical for a good performance.

4.6 Performance analysis on estimation of road surface temperature according to variance

Although a change in the collected road surface temperatures is not large, it can be somewhat large within a short period of time, as shown in data on Oct. 12 in 2016. The cause of this change in temperature can be estimated due to external environments (tunnels or bridges). Thus, it is necessary to evaluate a model that estimates a pattern of change in road surface temperature, in which the model is able to reflect such features appropriately. The (AAE-based) similarity of estimates of road surface temperature was analyzed additionally using road surface temperature samples whose variance was large through Model IV to evaluate explanatory power of the model on rapid pattern changes in road surface temperature quantitatively. The steps of analysis are as follows.

Step 1) Sample data are extracted from five road surface temperature data sets.
Step 2) The variance of the sample is calculated.
Step 3) Each of AAEs, with regard to estimated road surface temperatures and front and rear outdoor air temperatures, are calculated if the sample exceeds a certain level of variance.
Step 4) After Step 1–3 are conducted, these steps are iterated for another five sample data sets that are adjacent to the sample data.

The reason for the addition of Step 4 was to validate similarity of the estimated road surface temperature through the AAE when a level of variance in road surface temperature was large within the sample (that is, a temperature tended to increase or decrease or a change rate was large), and to compare it with the AAE of the front and rear outdoor air temperatures. Fig. 12 shows the results of variance calculated from each of the samples. In this analysis, the criterion of sample whose change in road surface temperature was large was set to 0.5°C based on Fig. 13. Several typical samples were investigated and the results showed that sample data whose variance was more than 0.5°C exhibited a clear pattern of rapid decrease or increase in temperature, whereas no such pattern was revealed in other sample data whose variance was less than 0.5°C. Thus, 0.5°C was selected based on empirical inference.

Fig. 13 shows AAE results between pattern of change in road surface temperature estimated based on samples whose variance was 0.5°C and the actual pattern of changes in road surface temperature. Fig. 13 visualizes the results of the above same calculation iteratively with regard to front and rear outdoor air temperatures. In Fig. 14, “○” refers to a
sample whose AAE based on the estimated road surface temperature was lower than AAE based on the outdoor air temperatures, and "■" refers to a sample whose AAE based on estimated road surface temperature was higher than the AAE based on the outdoor air temperatures. That is, the number of ○ is expected to be larger than that of ■ if the estimated road surface temperatures reflect a change in actual road surface temperature better than outdoor air temperature in a sample whose change in actual road surface temperature is large.

Fig. 14 Comparison AAE at highest Degree of Variance from Sample Data

The comparison results between rear outdoor air temperatures and estimated road surface temperatures in Fig. 13 (A) show that the number of ○ is 145 whereas the number of ■ is 19, resulting in a difference of 126. On the other hand, the number of ○ is 129 whereas the number of ■ is 35 in Fig. 13(B), resulting in a difference of 96. This result indicated that when a pattern of changes in road surface temperature exhibited high variance overall, the estimation of change in the road surface temperature using the model was more appropriate than using outdoor air temperature directly. Furthermore, this result verified indirectly that the front outdoor air temperatures reflected a pattern of change in road surface temperature better than the rear outdoor air temperature.

Model IV was constructed by integrating and utilizing road surface temperature and outdoor air temperature data only, collected in common times in 2015 and 2016. As a result, the performance of Model IV showed somewhat degraded estimation results as compared to the other three models. Although an amount of training data was increased by the integration of 2015 and 2016 data, ultimately, the deviation of the outdoor air temperatures used as training data to estimate a specific road surface temperature was large, resulting in the degradation of estimation performance of the model unexpectedly. Since environmental variables needed to be considered to reflect specificity properly to individual environments when data collected from various environments were integrated, the performances of the models were evaluated additionally in an environment whose change in road surface temperatures was large.

The present study analyzed quantitatively that the estimated road surface temperature using the model changed more sensitively than that based on the outdoor air temperatures, even in an environment where the actual change in road surface temperature was large.

5 CONCLUSION

The present study presented a methodology and models to estimate a pattern of changes in road surface temperature that can be foundational data to respond to road surface freezing preemptively, and establish strategies for ultimately the reduction in traffic accidents in winter. To estimate a pattern of changes in road surface temperatures, outdoor air temperatures were employed as a main predictor, and humidity and ambient temperature were also utilized as additional predictors. A road surface temperature acting as a response variable, was acquired using the thermal mapping system. The classification learner, which was a toolbox for the machine learning from MATLAB, was used for the modeling of a pattern estimation on changes in road surface temperature. The number of the machine learning algorithms provided by MATLAB was about 20, and the present study selected weighted KNN whose accuracy was the highest after executing all of them. The collected data of road surface temperatures were divided into training data for model construction and evaluation data of the model. Afterward, the models were constructed by using predictor and response variables for the training data in machine learning, and the model was evaluated using predictor and response
variables for the evaluation data. The inappropriate data in the model construction process were all removed through the outlier process. The four models constructed through the present study are presented in Table 2.

Table 2. Model Structure

<table>
<thead>
<tr>
<th>Variables</th>
<th>Model I</th>
<th>Model II</th>
<th>Model III</th>
<th>Model IV</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Humidity</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Temp.</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

The study results verified that the pattern of changes in road surface temperature estimated using the four constructed models was closer to the actual road surface temperatures than using outdoor air temperatures directly. The comparison of results between Model II, constructed by adding humidity and ambient temperature, collected newly from 2016, and Model III, using only outdoor air temperatures, showed that Model II had a slightly better explanatory power, although it was minimally better. This result implied that the consideration of additional predictors that can explain response variables other than outdoor air temperature can have a positive effect on the improvements of the model's performance. The evaluation results on Model IV, constructed with front and rear outdoor air temperatures, which were predictors in common in 2015 and 2016 showed performance degradation slightly compared to other models (Models I, II, and III). The data integration results of 2015 and 2016 showed that the absolute amount of training data was increased, but the heterogeneity between training data used to estimate a specific range of road surface temperature (15°C–20°C) was significantly large, which degraded the estimation performance. Changes in outdoor air temperatures that corresponded to similar road surface temperatures differed from year to year.

The present study qualitatively analyzed that the comparison of results between outdoor air temperature and estimated road surface temperature (Model IV), with regard to sample temperature data whose dispersion of actual road surface temperature was large in a short period of time, showed that the estimated road surface temperature was changed more sensitively according to changes in the actual road surface temperature. For the future study, it is necessary to classify a level of variables appropriately to improve performances of models that estimate a pattern of changes in road surface temperature using Classification Learner embedded in MATLAB. Currently, road surface and outdoor air temperatures are displayed to the tenth. If data can be classified using a criterion of 0.5°C considering the purpose of the data, it would be better to use the model after re-construction through data classification via data pre-processing. Furthermore, various environmental variables should be considered to construct a model that is sensitive to temperature changes according to physical changes (tunnels and bridges) or environmental changes (rapid change in temperature within the road section). In particular, a dummy predictor such as “data collection location” is needed to be added to the model to estimate temperature changes according to physical changes such as tunnels or bridges.

Conclusively, the most important factor for improvements on model performance is an amount of quality training data collected from various environments. That is, model performance will be improved if a combination of predictors that corresponds to the response variables is various and the amount of combination data is larger. The combination of currently collected response variables and predictors showed that most data was collected at a relatively stable environment whose change in temperature was within a narrow range of 13°C to 18°C. A model constructed with such an environment is likely to be vulnerable to the estimation of the road surface temperature in a heterogeneous environment. Thus, it is necessary to acquire quality data, collected iteratively, from various environments based on the proposed methodology in order to construct a model that can be utilized universally.

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REFERENCES

KEYWORDS:
Echelon Paving, Notched Wedge and Joint Heater, Longitudinal Joint Construction, Crack Seal

ABSTRACT:
To improve the performance of longitudinal joints in asphalt overlays DuPage Department of Transportation, IL (DuDOT) constructed pavements in various parts of the county using different construction techniques, namely notched wedge and joint heater, joint stabilization and echelon paving. Eight projects previously determined by DuDOT which included a total of 22 sample units were evaluated to compare the performance of the various construction methods. The condition of the joints constructed using the various methods were evaluated using detailed foot on ground surveys in one year, and manual survey of geo-referenced images in the following year. Shoulder joints in one particular project were designated as a control section for comparison. Results from this study indicated that notched wedge and joint heater and Echelon paving performed better than joint stabilization. In addition, the effect of crack seal on pavement condition was evaluated using the pavement condition data for three years.
1 INTRODUCTION

This paper presents the results of a study conducted to evaluate the various longitudinal joint construction techniques used by the DuPage Department of Transportation (DuDOT) in Illinois. In 2013, to improve the performance of longitudinal joints in asphalt overlays DuDOT constructed pavement in various parts of the county using different construction techniques, namely notched wedge and joint heater, joint stabilization and echelon paving. Eight projects selected by DuDOT were evaluated over two years through foot-on-ground field surveys in the first year (2014), and field imagery in the following year (2015). Shoulder joints in one particular project were designated as a control section for comparison. In addition, the effect of crack seal on pavement condition was evaluated by analyzing the condition data for three years. A brief literature review of longitudinal joint performance, and crack seal performance is presented below.

1.1 Longitudinal Joint Performance

The longitudinal joint is formed in asphalt pavements during construction of adjacent lanes. Proper construction of the longitudinal joints is required to reduce premature cracking, and raveling. Various methods have been employed to improve the construction of longitudinal joints including echelon paving, constructing wedge joints, using joint heater and stabilizers, and using a restrained edge (Nener-Plante 2012b). Several studies have been conducted to evaluate the performance of longitudinal joints using the various construction techniques (Huang et. al. 2010, Giehart 2012a, Nener-Plante 2012b, and Kandhal and Rao 1994)

Huang et. al. evaluated various joint construction techniques for asphalt pavements constructed in Tennessee (Huang et. al. 2010). The techniques were divided into the following categories:

- joint adhesives including an anionic emulsion and a polymerized emulsion commonly used by the Tennessee Department of Transportation (TDOT)
- a hot-applied high-polymer rubber, and a high-polymer emulsion
- joint sealers including a polymerized maltene emulsion
- polymerized agricultural oil
- infrared joint heater

Air void content, permeability, indirect tensile test (IDT), and water absorption tests were conducted using cores extracted from the field. The infrared heater was the best performing technique, and was effective in reducing the air void content, increasing permeability, and IDT strength.

Kandhal, and Rao conducted a research study to evaluate eight construction techniques on two sections of Hot Mix Asphalt (HMA) pavements in Michigan and Wisconsin (Kandhal and Rao 1994). The techniques included wedge joint with and without tack coat, rolling technique, and restrained edge compaction. Nuclear density and bulk specific gravity testing were conducted to verify the density values. Analysis results indicated that the wedge joint technique with and without tack coats yielded high density values.

A cooperative study to identify the best practices for construction of HMA longitudinal joints was conducted by the Federal Highway Administration (FHWA), and Asphalt Institute (AI) (Giehart 2012a). The study concluded that echelon paving was deemed best for constructing a longitudinal joint. However, this technique could be limited due to traffic considerations. Other preferred construction methods included notched wedge and traditional butt joint.

1.2 Crack Seal Performance

Crack sealing and filling is a preventive maintenance activity performed by most highway agencies in the United States of America (USA). Crack sealing and filling primarily focuses on preventing intrusion of moisture into the existing cracks. While crack sealing is used on working cracks (typically more than ¼ inch), crack filling is used for pavements that undergo little movement (Malen et.al. 2013 and Peshkin et. al. 2004a). Most available literature suggests that crack sealing is an effective pavement management technique (Malen et.al. 2013 and Peshkin et. al. 2004a). However, the effect of crack sealing on pavement performance, extension of life, and cost-effectiveness is still under study. Several agencies (Michigan, Plante 2013, 2018).
Texas, Wisconsin, and Ontario) have reported that crack sealing decreased the development of pavement distresses, and increased ride quality (Hand et.al. 2000).

A 10 year study was conducted to evaluate the prevailing crack sealing program for Ohio Department of Transportation through field evaluations of crack sealed and control sections (Rajagopal 2011). Results indicated crack sealing as an effective preventive maintenance tool for pavements with pavement condition index (PCI) in the range of 66 to 80. Performance prediction models indicated an increase in service life of up to 3.6 years with crack sealing.

Haddock et al., (Haddock et.al. 2004b) developed a statistical model to compare the performance between sealed and unsealed sections for HMA and composite pavements. Due to limited data (3 years), no significant conclusions were obtained on the cost-effectiveness of crack sealing pavements. However, results concluded no significant performance difference between sealed and unsealed sections during the project life of the HMA and composite pavements examined.

Two aspects of pavement maintenance; life expectancy of preventive maintenance treatments, and the timing were evaluated by Eltahan et al. (Eltahan et. al. 1999), using data from Long Term Pavement Performance (LTPP) Specific Pavement Study 3 (SPS-3) data. Results using survival analysis indicated that median survival times for thin overlays, slurry seal and crack seal were 7, 5.5, and 5.1 years respectively. In addition, chip seals performed better than thin overlays, slurry seals, and crack seals in controlling the reappearance of distress. A National Cooperative Highway Research Program (NCHRP) study conducted to evaluate optimum timing for preventive maintenance treatments indicate that there is a minor effect in terms of life extension when using crack sealing (Peshkin et. al. 2004a). Studies conducted as part of Michigan Department of Transportation (MDOT) reported 3 years of life expectancy on flexible pavements, as a result of crack sealing (Peshkin et. al. 2004a).

2 OBJECTIVE

The main objective of this study is to evaluate and compare over a period of time, the various construction methods selected to improve longitudinal joints in asphalt overlays and the effect of crack sealing on pavement performance.

3 CONSTRUCTION METHODS

DuDOT constructed eight projects with various techniques including notched wedge with joint heater, echelon paving, and notched wedge with joint heater and joint stabilizer/GSB88 to evaluate the performance of the longitudinal joint. The location of each project, along with the construction technique used is shown in Table 1.

Table 1. DuDOT’s eight projects for evaluating performance of longitudinal joint.

<table>
<thead>
<tr>
<th>Project</th>
<th>Construction Methods for longitudinal joint improvement</th>
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<tr>
<td></td>
<td>Method 1</td>
</tr>
<tr>
<td>1</td>
<td>Notched Wedge and Joint Heater</td>
</tr>
<tr>
<td>2</td>
<td>Notched Wedge and Joint Heater</td>
</tr>
<tr>
<td>3</td>
<td>Notched Wedge and Joint Heater</td>
</tr>
<tr>
<td>4</td>
<td>Notched Wedge and Joint Heater</td>
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<tr>
<td>5</td>
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</tr>
<tr>
<td>6</td>
<td>Notched Wedge and Joint Heater</td>
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<td>7</td>
<td>Notched Wedge and Joint Heater</td>
</tr>
<tr>
<td>8</td>
<td>Notched Wedge and Joint Heater</td>
</tr>
</tbody>
</table>

The following sections provide a brief description of the construction methods used for joint evaluation:
3.1 Notched Wedge and Joint Heater

This construction technique consists of a vertical notch and a wedge with a slope of 1:12 (vertical: horizontal) which is most commonly used. In order to accomplish this notched wedge for the first lane of paving, a device is attached to the paver extension to form it. For the second lane of paving an infrared heater in the front of the paver is used to heat the notched wedge joint for better adhesion, and to improve consolidation. Figure 1 shows a schematic of this construction technique.

![Notched Wedge Schematic](image)

Figure 1. A schematic of notched wedge joint.

3.2 Echelon Paving

Echelon paving involves paving multiple lanes at the same time with one paver closely following the paver in the adjacent lane. The idea behind this technique is to reduce the joint density differential as both mats are hot when they are compacted. However, this technique might not always be feasible due to traffic considerations. The echelon paving construction technique is shown in Figure 2.

![Echelon Paving Image](image)

Figure 2. Echelon paving (14).

3.3 Joint Stabilizer (Jointbond)

The joint stabilizer construction technique applies an emulsified material to the joint to help the bonding between mats and to avoid any water from entering the joint (12). The tack coat consists of asphalt cement, or emulsion and can be applied on any type of joint. Joint bond is an example of joint stabilizer and is applied one foot to one and half feet from each side of the joint, though the application rate might change with each manufacturer.

3.4 GSB-88

GSB-88 is a restorative sealer used for pavements in fair to good condition. This technique extends the life of the pavement by sealing the surface to repel water, thereby reducing surface raveling and premature...
cracking. This material can be applied to the surface of the pavement after construction to repel water and extend the life of the joint (13).

4 FIELD SURVEY METHODOLOGY

To evaluate the performance of longitudinal joints, a total of 22 tenth mile sample units were selected for field survey from the eight projects. The number of sample units selected for the survey were adjusted based on the length of each project, and is shown in Table 2. The locations of each of the 22 test sections were marked and recorded via perspective digital camera pictures to be used for future surveys. Shoulder joints in one particular project were designated as a control section for comparison.

Table 2. Number of random test sections per project, and Traffic Average Daily Traffic (ADT).

<table>
<thead>
<tr>
<th>Project</th>
<th>Estimated Length (mi)</th>
<th>Sample Units</th>
<th>Traffic ADT</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.58</td>
<td>4</td>
<td>6000</td>
</tr>
<tr>
<td>2</td>
<td>1.35</td>
<td>3</td>
<td>8400</td>
</tr>
<tr>
<td>3</td>
<td>0.46</td>
<td>2</td>
<td>29,200/43,100</td>
</tr>
<tr>
<td>4</td>
<td>0.51</td>
<td>2</td>
<td>22,600</td>
</tr>
<tr>
<td>5</td>
<td>0.64</td>
<td>2</td>
<td>16,000</td>
</tr>
<tr>
<td>6</td>
<td>0.93</td>
<td>3</td>
<td>20,200</td>
</tr>
<tr>
<td>7</td>
<td>0.92</td>
<td>3</td>
<td>23,700</td>
</tr>
<tr>
<td>8</td>
<td>1.00</td>
<td>3</td>
<td>28,000</td>
</tr>
</tbody>
</table>

The field survey methodology in 2014 included a detailed visual inspection of each sample unit and the joints within. The surveys focused primarily on cracking and raveling at each joint. The length and width of every crack identified was measured and recorded to determine the average width, and percent crack length in each sample unit. Additionally, the area of raveling identified was measured and classified as low, medium or high based on its severity. Illinois Department of Transportation (IDOT)’s condition rating system (CRS) methodology was used to provide a basis for rating the sample units (Heckel and Ouyang 2007).

As a second on-site survey element, an overall rating was provided for each sample unit on a scale of one to ten where 0 = unacceptable; 2 = poor; 4 = fair; 6 = good; 8 = very good; and 10 = excellent. The rating specific to the longitudinal joints, was used to assess the performance and variability of the longitudinal joint within individual sample units. The rating for each project is subjective, and was calculated based on the condition of the control section.

In 2015, the evaluation was conducted using field images collected using a Digital Survey Vehicle (DSV). Similar to 2014, the analysis consisted of assigning a rating to sample units on a scale of one to 10, by measuring distresses like cracking and raveling. As accurate estimation of crack width was not possible using the images, the severity of the cracks were classified as low, medium or high based on visual estimation of crack width.

5 CONSTRUCTION METHODS RESULTS

The consolidated results for joint cracking and rating of the longitudinal joints from the field surveys are shown below in Figure 3 and Figure 4, and Tables 3 and 4, respectively. Joint cracking results as shown in Figure 3 indicates that in 2014, sections with echelon paving outperformed the other construction techniques, followed by notched wedge and joint heater. However, in 2015, notched wedge and joint heater outperformed the other construction techniques, closely followed by echelon paving. In both the surveys, the highest amount of joint cracking was observed in the sections with joint bond and GSB 88. It is to be noted that most of the centerline joints were crack sealed prior to the 2015 survey, resulting in the reduction of joint cracking between 2014 and 2015. However, the control section indicated high amounts of joint cracking, with an increase in cracking from 2014 to 2015. Raveling was not observed in any of the sections surveyed.
As observed in Figure 4, the longitudinal joint ratings for 2014, and 2015 followed a trend similar to joint cracking. Sections with echelon paving, and notched wedge and joint heater performed better than the other techniques in 2014, and 2015, respectively. It should be noted that the results shown in the study are specific to the sections surveyed. Further research must be conducted prior to a consensus on the most appropriate construction technique for longitudinal joint performance.

Considering the fact that the field survey rating was restricted to the joints, the sample units were rated using the CRS methodology with the images collected in 2014 and 2015 as part of the pavement management system. The CRS survey provided an overall perspective on the performance of the sample units. Results of the CRS survey are shown in Figure 5.

Table 3. Construction methods results for 2014.

<table>
<thead>
<tr>
<th>Construction Method</th>
<th>Cracking</th>
<th></th>
<th>Raveling</th>
<th></th>
<th></th>
<th>Rating</th>
<th>St. Dev.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cracking Amount</td>
<td>% Cracks</td>
<td>Raveling</td>
<td>Length (ft)</td>
<td>Severity</td>
<td>St. Dev.</td>
<td></td>
</tr>
<tr>
<td>NW &amp; JH</td>
<td>205.33</td>
<td>188.23</td>
<td>43%</td>
<td>38%</td>
<td>NA</td>
<td>None</td>
<td>8.42</td>
</tr>
<tr>
<td>Echelon Paving</td>
<td>23.95</td>
<td>74.47</td>
<td>5%</td>
<td>14%</td>
<td>NA</td>
<td>None</td>
<td>9.63</td>
</tr>
<tr>
<td>NW &amp; JH &amp; GSB 88</td>
<td>361.42</td>
<td>97.51</td>
<td>68%</td>
<td>18%</td>
<td>NA</td>
<td>None</td>
<td>7.67</td>
</tr>
<tr>
<td>NW &amp; JH &amp; Jointbond</td>
<td>358.92</td>
<td>34.38</td>
<td>68%</td>
<td>7%</td>
<td>NA</td>
<td>None</td>
<td>8.00</td>
</tr>
<tr>
<td>Control Section</td>
<td>412.85</td>
<td>112.92</td>
<td>78%</td>
<td>21%</td>
<td>NA</td>
<td>None</td>
<td>6.42</td>
</tr>
</tbody>
</table>

Table 4. Construction methods results for 2015.

<table>
<thead>
<tr>
<th>Construction Method</th>
<th>Cracking</th>
<th></th>
<th>Raveling</th>
<th></th>
<th></th>
<th>Rating</th>
<th>St. Dev.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cracking Amount</td>
<td>% Cracks</td>
<td>Raveling</td>
<td>Length (ft)</td>
<td>Severity</td>
<td>St. Dev.</td>
<td></td>
</tr>
<tr>
<td>NW &amp; JH</td>
<td>7.94</td>
<td>15.87</td>
<td>2%</td>
<td>3%</td>
<td>NA</td>
<td>None</td>
<td>9.70</td>
</tr>
<tr>
<td>Echelon Paving</td>
<td>17.40</td>
<td>20.85</td>
<td>3%</td>
<td>4%</td>
<td>NA</td>
<td>None</td>
<td>9.57</td>
</tr>
<tr>
<td>NW &amp; JH &amp; GSB 88</td>
<td>21.67</td>
<td>20.21</td>
<td>4%</td>
<td>4%</td>
<td>NA</td>
<td>None</td>
<td>9.33</td>
</tr>
<tr>
<td>NW &amp; JH &amp; Jointbond</td>
<td>21.33</td>
<td>1.53</td>
<td>4%</td>
<td>0%</td>
<td>NA</td>
<td>None</td>
<td>9.50</td>
</tr>
<tr>
<td>Control Section</td>
<td>480.50</td>
<td>106.40</td>
<td>91%</td>
<td>20%</td>
<td>NA</td>
<td>None</td>
<td>6.58</td>
</tr>
</tbody>
</table>
Figure 3. Joint cracking percentage for the various construction techniques.

Figure 4. Rating for the various construction techniques.
6 CRACK SEAL STUDY

In addition to the construction evaluation study, the effect of crack seal on pavement performance was evaluated for the DuDOT pavement management sections. As part of the evaluation, the Maintenance and Rehabilitation (M&R) information from 2013 to 2015 was utilized to measure the effect of crack seal on pavement performance. Analysis included correlation of CRS deterioration with ‘crack sealed’ and ‘non-crack sealed’ pavements. The pavement sections with no ‘crack seal’ treatment performed during this period were included as the control samples, and sections which were rehabilitated during this time period were excluded from the analysis. No foot-on-ground surveys were performed for this task, and assessments were made using available imagery from 2013 to 2015.

The performance of a preventive maintenance technique depends on the age and current condition of the pavement. Based on reviewing the M&R treatment matrix for DuDOT, it was established that maintenance activities were performed for pavement sections with CRS greater than 6.5. A corresponding age was established for CRS value of 6.5 with information from the 2014 pavement performance model, and analyzed to include the effect of age (remaining life) due to crack seal. Based on this information, pavement sections were divided into four categories for analysis:

a) section with CRS less than or equal to 6.5
b) section with CRS greater than 6.5
c) section with age less than or equal to 8 years
d) section with age more than 8 years.

Table 5 shown below provides results of the statistical analysis of crack seal performance. Any improvement in average CRS, or age is denoted by a negative number.
Table 5. Statistical summary of crack seal performance

<table>
<thead>
<tr>
<th>Statistical Summary</th>
<th>CRS ≤ 6.5</th>
<th>CRS &gt; 6.5</th>
<th>Age &gt; 8</th>
<th>Age ≤ 8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total number of pavement segments</td>
<td>476</td>
<td>476</td>
<td>476</td>
<td>476</td>
</tr>
<tr>
<td>Total number of HMA surface segments</td>
<td>461</td>
<td>461</td>
<td>461</td>
<td>461</td>
</tr>
<tr>
<td>Total number of segments for analysis</td>
<td>425</td>
<td>433</td>
<td>425</td>
<td>433</td>
</tr>
<tr>
<td>Total number of segments in category</td>
<td>149</td>
<td>251</td>
<td>276</td>
<td>182</td>
</tr>
<tr>
<td>Number of control segments</td>
<td>135</td>
<td>249</td>
<td>187</td>
<td>153</td>
</tr>
<tr>
<td>Average change in CRS (Crackseal)</td>
<td>-0.29</td>
<td>0.60</td>
<td>0.73</td>
<td>0.52</td>
</tr>
<tr>
<td>Standard Deviation in CRS (Crackseal)</td>
<td>0.84</td>
<td>0.00</td>
<td>0.34</td>
<td>0.28</td>
</tr>
<tr>
<td>Average change in CRS (Control)</td>
<td>0.27</td>
<td>0.16</td>
<td>0.61</td>
<td>0.24</td>
</tr>
<tr>
<td>Standard Deviation in CRS (Control)</td>
<td>0.47</td>
<td>0.35</td>
<td>0.38</td>
<td>0.24</td>
</tr>
<tr>
<td>Average change in Age (Crackseal)</td>
<td>-1.88</td>
<td>1.65</td>
<td>1.09</td>
<td>1.95</td>
</tr>
<tr>
<td>Standard Deviation in Age (Crackseal)</td>
<td>4.10</td>
<td>0.00</td>
<td>1.50</td>
<td>1.44</td>
</tr>
<tr>
<td>Average change in Age (Control)</td>
<td>0.76</td>
<td>0.35</td>
<td>1.17</td>
<td>1.48</td>
</tr>
<tr>
<td>Standard Deviation in Age (Control)</td>
<td>1.79</td>
<td>1.13</td>
<td>1.41</td>
<td>1.76</td>
</tr>
</tbody>
</table>

From the results it is observed that for sections crack sealed in 2013, the average CRS increased by 0.29 between 2013 and 2014 for pavement sections with CRS less than or equal to 6.5. Similarly, for sections crack sealed in 2014, the average CRS decreased by 0.6 between 2014 and 2015 for pavement sections with CRS less than or equal to 6.5. However, for pavement sections with CRS greater than 6.5 the average change in CRS decreased from 2013 to 2014, and 2014 to 2015; indicating no effect of crack seal on pavement performance.

It was observed that, in most cases, crack sealing involved sealing the centerline joint only. Though the centerline crack seal is expected to increase the CRS rating, the general deterioration of the pavement associated with age will generate a counter effect leading to potential decrease in CRS. Overall, the results did not indicate any correlation between crack seal and increase in CRS. Future studies with more representative crack seal sections will be required to provide a definitive relationship.

Figures 6 and 7 shows progression of CRS from 2013 to 2015 for the pavement sections crack sealed in 2013 and 2014, respectively.
7 CONCLUSIONS AND RECOMMENDATIONS

To evaluate the performance of longitudinal joints in asphalt pavements, eight projects with various construction techniques were evaluated in this study. The condition of the joints constructed using the various methods were evaluated using detailed foot on ground surveys in 2014, and manual survey of georeferenced images in 2015. Shoulder joints in one particular project were designated as a control section for comparison.

Results indicated that in 2014, Echelon paving performed better than the other construction methods in terms of low percentage of cracking and high rating. However in 2015, the notched wedge and joint heater outperformed the other construction methods closely followed by the Echelon paving method. In 2015, most of the longitudinal joints were sealed, and deterioration of the joints between 2014 and 2015 were not captured accurately. The difference in performance between the various construction methods could be
attributed to difference in traffic Equivalent Standard Axle Loads (ESALs) and/or construction quality. Results from this study indicate that both echelon paving and notched wedge and joint heater work well for DuDOT. However, the greatest benefit to DuDOT will be realized through future evaluations, which will provide information on performance of each construction method over time. Future years of evaluation will provide an opportunity to conduct a comparison of findings that can include a benefit cost analysis to estimate the impact of each treatment method.

In addition, the effect of crack seal on pavement condition was evaluated using the pavement condition data for three years. Results from this study were inconclusive on the effect of crack sealing on pavement performance, with respect to change in CRS values. As crack sealing was restricted to the centerline joints, deterioration of other distresses contributed to increase in CRS values in most cases. In general, crack sealing prevents intrusion of water into the pavement structure, improves pavement performance and life, and decreases the development of other distresses. Sealing all possible cracks on the pavement surface will provide a possible benefit to the CRS values.
REFERENCES

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<td>Sustainable Flexible Pavement Design Using Recycled Plastic for Bikeways in Ecuador</td>
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<td>MOYANO, Christian</td>
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**KEYWORDS:**
- PET plastic
- Modified asphalt mixes
- Life cycle assessment
- Recycling sustainability.

**ABSTRACT:**
On recent years, Cuenca – Ecuador showed substantial increases in its populations and motor vehicles count, negatively leading towards increments on environmental pollution and emissions; because of that, and under the purpose of seeking for a sustainable city, local transportation agencies are working hard with the aim of improving its mobility planning and management for all transportation modes, including bicycles. Consequently, an important part of this strategy involves the design and implementation of a significant bikeway network throughout the city, however, the implementation of this bikeway network may represent environmental pollution itself if constructed with traditional flexible pavements.

To avoid this potential problem, the objective of this paper is to present an alternative flexible pavement design that is more sustainable by incorporating recycled plastic in the mixture, and at the same time, maintaining the required performance. This research develops the design of a modified asphalt incorporating in the mixture crushed PET plastic that otherwise will become waste. This alternative asphalt will comply with construction codes for overlay mixes that will be placed on defined cycle paths and public spaces in Cuenca city.

The design process involves Marshall, abrasion, VA, flow and stability tests over different mixes (with eight different PET plastic percentages) following Spanish standards to analyze its performance. Finally, life cycle assessment (LCA) is also performed to define improvements from a sustainable perspective.
Sustainable Flexible Pavement Design Using Recycled Plastic for Bikeways.

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1. INTRODUCTION

Plastic is an artificial product obtained via chemical reactions for no-resin matter according to “Aplicaciones del Plástico en la Construcción”, plastic products are one of the most used alternative products on flexible pavements in Europe. Polyvinyl chloride (PVC) ranks as the most common material used in flexible pavements (de Cusa, 1977), (Modarres & Hamedi, 2014a).

Another alternative material used as addition in pavement asphalt mixtures is Polyethylene terephthalate (PET); its main used is the elaboration of bottles. (Modarres & Hamedi, 2014b, Widojoko & Purnamasari, 2012) Its principal advantages are:

- Easy fabrication
- Reduced thermal sensibility for the molten material
- High tenacity
- Good flexibility
- Insensitive to cracking
- Chemical products resistance
- High strength
- Low absorption

Several studies about its use as an alternative material has been conducted, comparing performances, not only with this addition, but also with fibers, glass, etcetera; findings are promising, for example there is evidence of increase in adherence between asphalt bitumen and aggregates (Xu et al, 2010).

Other research shows improvements in deformation characteristics, and fatigue resistance, their conclusions show that there is a better fatigue resistance performance when crushed glass material is added in the mixture (Arabani M et al, 2010).

Qunshan et al., 2009 made research with polyester, and mineral fibers stablishing optimum percentages of these additions around 3 to 4 % of the total mix weight, improving fatigue resistance.

A. Modarres & H. Hamedi, 2014a compared traditional asphalt mixes with proposed asphalt mixes containing 5.7% of PET obtaining Elastic modulus models and fatigue models. The main tests included indirect tensile testing (ITS), showing an increase in the tensile strength for the proposed mixture for different temperatures (5 and 20°C), but the elastic modulus decreased for the proposed mixture.

Similar studies made by Baghaee Moghadam et al, 2012 incorporating 0.2% of PET in the mixture determined a small increase in the stiffness. However, when increasing PET percentage this stiffness values started to decreased significantly (Modarres & Hamedi, 2014a).

A. Modarres & H. Hamedi, 2014b, in its second study proceeded with the same range for PET and asphalt, but in this case, they followed ASTM: D4123 and EN 12697-24 for the Elastic modulus tests, and fatigue tests, respectively. Their findings show that when increasing PET over 2%, the elastic modulus was reduced for both tested temperatures (5 and 20 °C) (Modarres & Hamedi, 2014b).
In 2012, Moghaddam et al analyzed stiffness and fatigue with mixtures containing 1% of PET and traditional mixtures, observations concluded that durability was indeed increased twice, but stiffness decreased slightly (Ahmadinia et al, 2011).

According to Widojoko & Purnamasari, 2012, density is negatively affected when PET is introduced. However, for additions between 0 to 4%, cohesion and adherence properties were improved, (Widojoko & Purnamasari, 2012).

For this study, taking into account that bikeways take relatively smaller loads and load configurations, the focus is more on safety and ride quality, therefore, durability and stability testing are proposed. With a strong focus on sustainability.

Even though PET does not improve asphalt pavement performance significantly, it can be used due to its low cost as a sustainable material. Vázquez, 2010 states that PET gives lower viscosity, better flexibility at low temperatures, and better rutting performance.

With this summary and scope defined, the following objectives are drawn.

**General Objective**

Design a PET-modified asphalt mix to be applied over bikeways that represents a positive alternative, including sustainable approaches by reducing environmental impacts, while maintaining the same or better performance than the base mix.

**Specific Objectives**

- Determine the optimum PET percentage that the mixture performs with the least abrasion and higher stability.
- Analyze durability, by determining the abrasion reduction percentages for all proposed mixes vs. the traditional standardized mixture.
- Verify standards compliance for all analyzed mixtures, especially for the chosen mix built with the optimum PET percentage.
- Propose the final modified mix design that complies with standards, while improving sustainability factors like energy consumption, and CO₂ emissions.

2. **METODOLOGY**

Among various tests defined below, the most representative performance characteristic to be analyzed is abrasion, Cantabrian tests is one of the most common method to do so, where a Marshall-type briquette at a constant temperature (between 15 & 30 °C) is introduced in the Los Angeles testing machine without the steel spheres, following the NLT-159, Resistance to plastic deformation of bituminous mixtures using Marshall equipment (Vilaplana, 2015).

This study was conducted using crushed PET with particles between 3-5 mm.

**A. Mix design**

Four aggregate types were used for the mixing process; crushed material passing sieve 3/4, crushed material passing sieve 3/8, crushed material passing sieve 3/16, and river sand. All materials come form Guachapala-Azuay, and Cochancay-Cañar (recognized and approved mines and quarries in Ecuador).

The public agency ASFALTAR EP (one of the approved public agencies for Cuenca City) provided all materials; moreover, the base/traditional mix design complied with the standardized design used by this public agency.

Each and every one of the materials were completely characterized for parameters like granulometry, absorption, specific gravity (Net, SSD, apparent); results are shown in table 1.

The ½” standardized granulometry used for this study is presented in table 1 below and is described on “Especificaciones Generales para la Construcción de Caminos y Puentes”. The ASTM C 136 and AASHTO T27 standards were strictly followed.
Table 1. Standardized Granulometry. (MOP, 2002, page IV 95).

<table>
<thead>
<tr>
<th>SIEVE PERCENTAGE BY WEIGHT PASSING (SQUARE MESH)</th>
<th>3/4&quot;</th>
<th>1/2&quot;</th>
<th>3/8&quot;</th>
<th>No4</th>
</tr>
</thead>
<tbody>
<tr>
<td>1&quot; (25.4 mm)</td>
<td>100</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>3/4&quot; (19.0 mm)</td>
<td>90-100</td>
<td>100</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>1/2&quot; (12.7 mm)</td>
<td>--</td>
<td>90-100</td>
<td>100</td>
<td>--</td>
</tr>
<tr>
<td>3/8&quot; (9.50 mm)</td>
<td>56-80</td>
<td>--</td>
<td>90-100</td>
<td>100</td>
</tr>
<tr>
<td>No4 (4.75 mm)</td>
<td>35-65</td>
<td>44-74</td>
<td>55-85</td>
<td>80-100</td>
</tr>
<tr>
<td>No8 (2.36 mm)</td>
<td>23-49</td>
<td>28-58</td>
<td>32-67</td>
<td>65-100</td>
</tr>
<tr>
<td>No16 (1.18 mm)</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>60-80</td>
</tr>
<tr>
<td>No30 (0.60 mm)</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>25-65</td>
</tr>
<tr>
<td>No50 (0.30 mm)</td>
<td>5-19</td>
<td>5-21</td>
<td>7-23</td>
<td>7-40</td>
</tr>
<tr>
<td>No100 (0.15 mm)</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>3-20</td>
</tr>
<tr>
<td>No200 (0.075 mm)</td>
<td>2-8</td>
<td>2-10</td>
<td>2-10</td>
<td>2-10</td>
</tr>
</tbody>
</table>

Specific gravities and absorption determination were performed following the ASTM C 128 – AASHTO T-84; results are shown in Table 2 below.

Table 2. Specific weight & Aggregate absorption

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>2.732</td>
<td>2.751</td>
<td>2.785</td>
<td>0.70</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>2.684</td>
<td>2.717</td>
<td>2.777</td>
<td>1.24</td>
</tr>
<tr>
<td>3/16&quot;</td>
<td>2.683</td>
<td>2.705</td>
<td>2.743</td>
<td>0.80</td>
</tr>
<tr>
<td>River sand</td>
<td>2.674</td>
<td>2.717</td>
<td>2.795</td>
<td>1.62</td>
</tr>
</tbody>
</table>

B. Marshall Test methodology

Marshall Test were used to determine the optimum percentage for asphalt, this percentage was determined after fulfilling the specification range criteria for the aggregates, Table 3 illustrates this standardized specification along with the chosen percentages for each aggregate in order to meet this criterion. Fig 1. Depicts these relationships.

Table 3. Aggregate combination chosen for the mix design.

<table>
<thead>
<tr>
<th>SIEVE OPENING</th>
<th>PERCENTAGES IN WEIGHT PASSING</th>
<th>MIXTURE 100%</th>
<th>SPECIFICATION RANGE 1/2&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>10,0%</td>
<td>22%</td>
<td></td>
<td>100</td>
</tr>
<tr>
<td>CRUSHED</td>
<td>CRUSHED</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MATERIAL</td>
<td>MATERIAL</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>99,69</td>
<td>100,00</td>
<td>100,00</td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>48,94</td>
<td>100,00</td>
<td>100,00</td>
</tr>
<tr>
<td>No. 4</td>
<td>0,34</td>
<td>27,99</td>
<td>98,75</td>
</tr>
<tr>
<td>No. 8</td>
<td>0,00</td>
<td>5,46</td>
<td>79,73</td>
</tr>
<tr>
<td>No. 50</td>
<td>0,00</td>
<td>0,00</td>
<td>28,47</td>
</tr>
<tr>
<td>No. 200</td>
<td>0,00</td>
<td>0,00</td>
<td>13,44</td>
</tr>
</tbody>
</table>

lower limit | Upper limit |
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>90</td>
<td>100</td>
</tr>
<tr>
<td>44</td>
<td>74</td>
</tr>
<tr>
<td>28</td>
<td>58</td>
</tr>
<tr>
<td>5</td>
<td>21</td>
</tr>
<tr>
<td>2</td>
<td>10</td>
</tr>
</tbody>
</table>
Fig. 1. Aggregates combined granulometric curve.

Results for Marshall Test varying percentages from 5.5 to 7.5 % are detailed in Table 4.

Table 4. Marshall Test for base mix design

<table>
<thead>
<tr>
<th>% ASPHALT</th>
<th>STABILITY</th>
<th>NET SPECIFIC WEIGHT</th>
<th>FLOW</th>
<th>Volume filled with asphalt (VFA)</th>
<th>% VOIDS (VA)</th>
<th>Volume of mineral aggregate (VMA)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.5</td>
<td>3059.102</td>
<td>2.279</td>
<td>11.333</td>
<td>49.301</td>
<td>9.940</td>
<td>19.605</td>
</tr>
<tr>
<td>6.0</td>
<td>3415.865</td>
<td>2.331</td>
<td>10.933</td>
<td>63.494</td>
<td>6.708</td>
<td>18.374</td>
</tr>
<tr>
<td>6.5</td>
<td>3021.174</td>
<td>2.337</td>
<td>12.200</td>
<td>69.900</td>
<td>5.513</td>
<td>18.314</td>
</tr>
<tr>
<td>7.0</td>
<td>3366.005</td>
<td>2.343</td>
<td>15.100</td>
<td>74.292</td>
<td>4.755</td>
<td>18.495</td>
</tr>
<tr>
<td>7.5</td>
<td>3153.588</td>
<td>2.333</td>
<td>15.333</td>
<td>82.312</td>
<td>3.372</td>
<td>19.064</td>
</tr>
</tbody>
</table>

By plotting percentage of asphalt vs stability, net specific weight, flow, VFA, VA, and VMA; the optimum asphalt percentage was determined, being 6.7%. This value will be used for both all upcoming mixes, base mixes, and mixes with PET additions.

C. PET addition on Base mix

For traditional design, materials are warmed up to 160 °C (320 °F), for the mixing process. But, when using PET, this material starts to molten, generating lumps and clots affecting homogeneity.

From several testing during this study, the recommended temperatures for adding PET in the mix is 110°C (230 °F), same recommendations can be found in (Cavanzo & Ortiz, 2002) study, where the recommendation is 110 and 120 °C (230 and 248 °F).

However, for the final stages of this research, the chosen mixing process was to add PET by following the methodology named “proceso en seco” where PET is first mixed with the aggregates, then these initial mix is warmed up to 130 °C (266°F), to finally be joined with the hot asphalt (Modarres & Hamedi, 2014b).

Briquettes were prepared according to Marshall Procedures using the following PET percentages: 2, 3, 4, 5, 6, 8, 10 & 12%. Findings show that flow increases when PET percentage increases; likewise, Stability increases first and then decreases when PET percentage increases (peak stability is around 6% PET); more details are shown and discussed below.

D. Abrasion Test

Following the NLT-352/86 standard, Characterization of the Open Bituminous Mixtures by Means of the Cantabrian Test of Loss by Wear; abrasion was measured for the dry briquettes just after weighting them, with speeds within 3.1 to 3.5 rad/s during 300 revolutions.
After this process, the briquettes were weighted again. A number of four briquettes for each PET percentage were tested. Loss due to abrasion was measured as follows

\[ W = \frac{W_1 - W_2}{W_1} \times 100 \]  
Eq. 1

Where

- \( W \) = loss due to abrasion (%)
- \( W_1 \) = initial briquette mass (g.)
- \( W_2 \) = final briquette mass (g.) (NLT-352/86)

3. RESULTS AND DISCUSSION.

Briquettes with added PET showed more Va in general as shown in a couple of examples in Fig. 2. Briquettes with contents of 0, 6 and 12% PET were compared; each with Va values of 4.79, 13.97 and 21.05% respectively.

For Marshall testing, briquettes with PET additions from 2% to 6% showed a sustained growth in stability, but over 6% showed a sudden decrease in their stability. Fig. 3 shows the peak value for stability and its correspondent PET percentage. NOTE: Since there was a problem with briquettes at 4%, this particular value was not included in the trend.

With respect to Flow, a proportional growth was found when PET percentages increase for all the range chosen for the study (from 2 to 12%), Fig 4 shows this trend below.

\[ \text{Fig. 2. Comparative for asphalt briquettes. Upper images: 0% PET (Va= 4.79%) vs. 6% PET (Va= 13.97). Lower images 0%PET (Va= 4.79%) vs. 12% PET (Va= 21.05%).} \]
Based on Marshall Results, 6% was the chosen optimum percentage for PET; since this value represents the closest real point to the maximum stability, notice than this value is similar to the recommended by previous literature cited before (5.7% as optimum PET percentage).

Further research made for the base mix and the proposed mix with the chosen optimum PET percentage was developed. RICE testing was made for both in order to find the theoretical maximum specific gravity (Gmm); results showed that for the base mix, Gmm equals 2.451, and for the proposed mix Gmm equals 2.375, with Va = 4.79% and Va = 13.97% respectively, validating the higher porosity for the proposed mix (as shown in fig. 2 above).

Abrasion testing was made over four briquettes with 0% PET and over four briquettes with 6% PET (chosen optimum percentage), using Eq. 1, and results are depicted in table 5.

**Table 5. Abrasion results.**

<table>
<thead>
<tr>
<th>% PET</th>
<th>W1 (g)</th>
<th>W2 (g)</th>
<th>% ABRASION</th>
<th>VARIANCE</th>
<th>STANDARD DEVIATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0%</td>
<td>1170.34</td>
<td>1104.79</td>
<td>5.60</td>
<td>7.74</td>
<td>1.88</td>
</tr>
<tr>
<td></td>
<td>1108.24</td>
<td>1001.89</td>
<td>9.60</td>
<td>3.54</td>
<td>1.88</td>
</tr>
<tr>
<td></td>
<td>1194.43</td>
<td>1113.84</td>
<td>6.75</td>
<td>7.74</td>
<td>1.88</td>
</tr>
<tr>
<td></td>
<td>1171.27</td>
<td>1065.68</td>
<td>9.02</td>
<td>3.54</td>
<td>1.88</td>
</tr>
<tr>
<td>6%</td>
<td>1082.84</td>
<td>922.2</td>
<td>14.84</td>
<td>15.36</td>
<td>7.85</td>
</tr>
<tr>
<td></td>
<td>1160.67</td>
<td>1072.69</td>
<td>7.58</td>
<td>61.65</td>
<td>7.85</td>
</tr>
<tr>
<td></td>
<td>1173.78</td>
<td>1023.22</td>
<td>12.83</td>
<td>61.65</td>
<td>7.85</td>
</tr>
<tr>
<td></td>
<td>1048.25</td>
<td>773.5</td>
<td>26.21</td>
<td>61.65</td>
<td>7.85</td>
</tr>
</tbody>
</table>
This outcome highlights that the proposed mix design doubles the abrasion percentage, however, the abrasion percentage for the proposed mix complies with the standard (abrasion must be lower than 25% according to the NLT-352/86 standard), fig. 5 shows a comparison of the abrasion rate for this test.

![Fig. 5. Abrasion results comparison, for 6% PET (left) vs. 0% PET (right)](image)

Finally, with the help of a sustainable calculator software like PaLATE, and considering maintenance every 5 years, the environmental impact for both design mixes (base and proposed) was determined, in terms of energy consumption, and CO2-e emissions.

For this determination, several factors were considered, like equipment, extraction, material transport, etc. Fig. 6 and Table 6 shows this final comparison.

The exercise was made for 1 km of bikeway, with 0.2 m thickness, and two ways of 1.8 m width in total. Considering the same volume of pavement mix for both designs (base mix and proposed 6% PET mix).
Notice that, for both, energy consumption and CO2-e emissions, there is a considerable improvement in terms of sustainability, a very significant factor when proposing alternative materials for future design. As mentioned before, these results are also presented as matrix format in table 6 below for better numerical comparisons.

**Table 6. Sustainability results matrix**

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Base mix</th>
<th>Proposed 6% PET mix</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Energy [GJ]</td>
<td>CO2e [kg]=GWP 0% PET</td>
</tr>
<tr>
<td></td>
<td>0% PET</td>
<td>6% PET</td>
</tr>
</tbody>
</table>

| Initial Construction     | INITIAL Materials Production | 1,112.2 | 614,706.0 | 1,064.7 | 598,471.2 |
|                         | INITIAL Materials Transportation | 334.7 | 23,090.6 | 316.9 | 21,859.1 |
|                         | INITIAL Equipment           | 3.6 | 246.2 | 3.5 | 240.3 |

| Maintenance             | MAINTENANCE Materials Production | 1,110.5 | 614,009.8 | 1,064.7 | 598,485.1 |
|                         | MAINTENANCE Materials Transportation | 1,670.8 | 115,267.4 | 1,584.3 | 109,299.4 |
|                         | MAINTENANCE Equipment          | 3.6 | 245.9 | 3.5 | 240.3 |

| Total                   | TOTAL Materials Production | 2,222.7 | 1,228,715.8 | 2,129.5 | 1,196,956.3 |
|                         | TOTAL Materials Transportation | 2,005.5 | 138,358.0 | 1,901.2 | 131,158.5 |
|                         | TOTAL Equipment               | 7.1 | 492.2 | 7.0 | 480.6 |

| Total                   |                      | 4,235.4 | 1,367,566 | 4,037.6 | 1,328,595 |

4. **CONCLUSIONS**

It was possible to propose a more sustainable pavement mix design for bikeways that complies even with current normative and standards for high volume traffic, current standards that are established by the “Ministerio de Transporte y Obras Públicas Ecuador (MTOP),”

The optimum PET percentage was determined to be 6%, considering as the percentage related with the maximum stability value, complying as well with Flow parameters.
Even though, the traditional base mix presents significantly better abrasion resistance (almost twice the proposed value), both mix designs comply with the standards comfortably.

The 6% PET mix stability value of 1752 lb. complies even with the motor vehicles traffic standards, for the “low traffic” category (category that has significantly higher traffic, and significantly heavier traffic than the expected traffic for bikeways).

From the sustainability analysis, it is important to conclude that, besides proposing a more economic mix design that complies with current standards, it is also a more sustainable mix design in terms of environmental impact. The proposed mix design uses less energy during its construction and maintenance and produces less CO2-e emissions than the traditional base mix design used nowadays.

Finally, it is important to mention that all municipal, public and private agencies in Ecuador can use this mix design not only for bikeways, but also for all their walkway projects in parks, trails, recreational areas, etcetera. Given that the proposed mix design complies with strict standards satisfactorily, with the additional benefits of being a cheaper and more sustainable pavement mix design.

REFERENCES.

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NLT-159 «Resistencia a la deformación plástica de mezclas bituminosas empleando el aparato Marshall».


Especificaciones generales para la construcción de caminos y puentes. (2002). Ministerio de Obras Públicas y Comunicaciones (República del Ecuador)


NLT-352/86 «Caracterización de las mezclas bituminosas abiertas por medio del ensayo cántabro de pérdida por desgaste». 
Research on Prolonging the Pavement Life with Advanced Polymer Modified Asphalt

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KEYWORDS:
Polymer Modified Asphalt, Pavement life, repair and rehabilitation,Durability, Reflection crack

ABSTRACT:
Based on the recently research for a large-scale survey on the express highway in service in Japan, it was essential to prolong the pavement life as following countermeasures, (1) to prevent penetration of rainwater from the road surface, (2) to remain the interlayer adhesive strength between asphalt layers (surface and binder course), (3) to reserve fatigue resistance and the thickness of asphalt base course, and (4) to take measures against stagnation of rainwater in the subbase course.

These means that it is important not to allow rain water to penetrate into the pavement in heavy traffic roadways, or to properly drain the permeated water, thereby maintaining the bearing capacity of the sub-base layer in a sound state which is important for prolonging the life of pavement surface.

Therefore, as one measure to prolong the asphalt pavement life, the advanced polymer modified asphalt was developed not only to improve the rutting resistance, but also to suppress the cracking in the asphalt mixture layer as much as possible. This asphalt mixture is expected to prevent the entry of water into the pavement below the subbase course and to suppress the cracking.

Through this research, the laboratory test and trial construction for the developed modified asphalt mixture were conducted to confirm the effect of the presence of rainwater intrusion into the subbase course.

In this paper, it is reported that the effect of rainwater infiltration on the bearing capacity of the pavement, and also the evaluation on constructability through the trial construction.
Research on Prolonging the Pavement Life with the Advanced Polymer Modified Asphalt

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1 INTRODUCTION

The “Pavement Inspection Procedure” established by the Bureau of Public Roads of the Ministry of Land, Infrastructure, and Transport in October 2016 clearly stipulates that pavement inspection shall be done based on three criteria—namely, “crack rate,” “rutting amount,” and “IRI”—and the target years of use shall be determined to protect the layers below the roadbed in order to prolong the service life. In a report of Takahashi et al., the results of a large-scale open-cut survey on an in-service highway route clarify the damage situation of asphalt pavement during long-term service amidst heavy traffic, including fatigue cracks and permanent deformation, embrittlement due to interlayer adhesion, and roadbed water-bearing. Also, based on these actual damage situations, the following measures to prolong service life are noted to be important: i) countermeasures against penetrating rainwater from the road surface, ii) ensuring interlayer adhesion of the asphalt mixture, iii) ensuring fatigue countermeasures and the layer thickness of the asphalt base (upper layer roadbed), and iv) countermeasures against water bearing in the roadbed.

This indicates that, particularly amidst heavy traffic, keeping rainwater and other water from penetrating the pavement body or properly draining the penetrated water in order to soundly maintain the bearing capacity below the roadbed is one important measure for prolonging the service life of asphalt pavement.

Therefore, as a measure to prolong the service life of asphalt pavement, the authors endeavored to suppress cracking in the asphalt mixture layer as much as possible and to prevent water penetration into the pavement body below the roadbed. To this end, we developed modified asphalt, which is excellent in terms of suppressing cracking. More specifically, first, we experimentally verified the influence of rainwater penetration into the roadbed from cracks on the pavement's bearing capacity, and we confirmed the importance of suppressing cracking. In addition, we sorted out a variety of factors related to cracking and the performance required to act as a countermeasure against such factors, and we attempted to develop the advanced modified asphalt that could satisfy the required performance criteria.

In this paper, we report the results of experimental confirmation of the influence of rainwater penetration into the roadbed on pavement bearing capacity as well as the results of the property test of the modified asphalt we developed and the mixture using the advanced modified asphalt, including its evaluation results regarding workability by in-house trial construction and so forth.

2 OVERVIEW OF STUDY

2.1 Confirmation of the influence of water penetration into the pavement body on pavement bearing capacity

At the pavement driving experiment facility of the Civil Engineering Research Institute of the National Research and Development Corporation, i) a road surface section where minor cracks had occurred and ii) a section where minor cracks had been repaired (sealed) with thin overlay cutting were prepared. A loading car was driven on the section for 10 months; thus, the effects of rainwater penetration into the pavement body on the pavement bearing capacity due to the repeated load of the transportation vehicle was verified by the change in the distortion amount by the FWD.

2.2 Development of modified asphalt

The causes of cracking in the asphalt pavement were sorted out by form using the Maintenance Guidebook for Road Pavement 2013 Edition and other references. The required performance and evaluation items for the asphalt and asphalt mixture were then set.

During the development of the asphalt, a plurality of modifying agents were selected, and evaluation tests of the binder properties and asphalt mixture properties of prototypes that adjusted the combination and mixing ratio were carried out. Thus, a modified asphalt that satisfied the established required performance criteria was developed.
3. SURVEY IN A LARGE SCALE TEST

3.1 Test method for survey

At the pavement driving experiment facility of the Civil Engineering Research Institute of the National Research and Development Corporation, a test in which a loading car was driven for 10 months on the two adjacent road surfaces described below was carried out.

i) A road surface section in which minor cracks had occurred (hereinafter, the "comparative section")

ii) A section in which minor cracks had been repaired (sealed) with thin overlay cutting (hereinafter, the "rainwater countermeasure section")

The pavement driving experiment facility is a circular circuit with a width of 3.2 m and a length of 628 m upon which an unmanned loading car runs. Photo 1 shows the circumstances of the accelerated load test by the loading car. In the experiment, linear transverse cracks having a width of about 0.2 to 0.4 mm occurred at a crack ratio of approximately 5 to 10%, and the length of each section was set to 16 m (32 m in total).

The loading car was made to run 45,000 times for 10 months (corresponding to 2.7 years with N5 traffic). The amount of distortion by the FWD before and after the loading car was driven were measured, and the influence of rainwater and the repeated load of the transport vehicle on the pavement bearing capacity were confirmed. This verification is part of joint research with the Public Works Research Institute.

![Photo 1 Circumstances of the accelerated load test](image)

3.2 Test results

The rate of increase in the distortion amount after 10 months of driving the loading car compared to the distortion before driving \( \left\{ \frac{\text{distortion amount after driving 10 months} - \text{distortion amount before driving}}{\text{pre-running distortion amount}} \times 100 \%(\%) \right\} \) is shown in Figure 1. The rate of increase in the \( D_0 \) distortion amount after driving the loading car was 9.0% in the comparative section and 4.3% in the rainwater countermeasure section. The rate of increase in the distortion of \( D_{300} \) was 15.2% for the comparative section and 9.4% for the rainwater countermeasure section.

From these results, it can be seen that the rate of increase in both the \( D_0 \) and \( D_{300} \) distortion amounts of the rainwater countermeasure section are smaller than those of the comparative section, so it was confirmed that reduction in pavement bearing capacity can be diminished by preventing the penetration of rainwater and so forth into the pavement.

From this, it was confirmed that suppressing cracking in the asphalt mixture layer of the surface layer or the surface/base layer is important to maintain the pavement bearing capacity—in other words, to prolong the service life. For reference, the number of days when rainfall of 1 mm or more occurred during the running experiment period of 10 months (February 10, 2015 to December 12, 2015: 305 days) was 94 days, the average rainfall per day on those days was 14.4 mm, and the total rainfall during the experiment period was 1,351 mm.\(^5\)
4. DEVELOPMENT OF THE ADVANCED MODIFIED ASPHALT

4.1 Required performance and evaluation items

Regarding the form of cracks that occurred on the asphalt pavement, the required performance and evaluation items for the asphalt and asphalt mixture were sorted out based on occurrence factors. The results are shown in Table 1.

The evaluation tests of the modified asphalt developed based on Table 1 were the (1) Fraass embrittlement point (embrittlement point), (2) bending test (bending strain), (3) DSR test (G*sinδ, G*/Sinδ), and (4) BBR test (S value, m value).

In addition, the tests for confirming the performance of the asphalt mixture were the (1) wheel tracking test (dynamic stability), (2) bending test (embrittlement point, bending strain), (3) bending fatigue test (cycles to failure), and (4) temperature stress test (stress relaxation limit point).

The target performance value was determined to be superior in all evaluation tests to polymer modified Asphalt Type II (hereinafter, "Modified Type II").

### Table 1  Required performance and evaluation items for the asphalt and mixture corresponding to the damage forms

<table>
<thead>
<tr>
<th>Damage form</th>
<th>Cause</th>
<th>Main test - Required performance - Index</th>
<th>Asphalt - Test method</th>
<th>Required performance - Index</th>
<th>Mixure - Test method</th>
<th>Required performance - Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fatigue cracks</td>
<td>Repeated tensile strain on the mixture layer's undersurface</td>
<td>DSR test</td>
<td></td>
<td>Cracking resistance → G*sinδ</td>
<td>Bending test</td>
<td>Fatigue resistance → Cycles to failure</td>
</tr>
<tr>
<td>Top-down cracks</td>
<td>Repeated tensile strain on the mixture layer's top surface</td>
<td>DSR test</td>
<td></td>
<td>Cracking resistance → G*sinδ</td>
<td>Bending test</td>
<td>Fatigue resistance → Cycles to failure</td>
</tr>
<tr>
<td>Opening of</td>
<td>Accumulation of mixture shrinkage</td>
<td>BBR test</td>
<td></td>
<td>Stress relaxation property → S value, m value</td>
<td>Stress relaxation property → Stress relaxation limit point</td>
<td></td>
</tr>
<tr>
<td>construction joints</td>
<td>Repeated expansion and contraction due to temperature changes</td>
<td>Fraass embrittlement point test</td>
<td></td>
<td>Flexibility at low temperatures → Fraass embrittlement point</td>
<td>Stress relaxation property → Stress relaxation limit point</td>
<td></td>
</tr>
<tr>
<td>Temperature</td>
<td>BRR test</td>
<td>Stress relaxation property → S value, m value</td>
<td></td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Temperature stress</td>
<td>Repeated expansion and contraction due to temperature changes</td>
<td>BRR test</td>
<td></td>
<td>Stress relaxation property → Stress relaxation limit point</td>
<td></td>
<td></td>
</tr>
<tr>
<td>REFRACTION cracks</td>
<td>Cracking of the construction base</td>
<td>DSR test</td>
<td></td>
<td>Cracking resistance → G*sinδ</td>
<td>Bending test</td>
<td>Fatigue resistance → Cycles to failure</td>
</tr>
<tr>
<td>Crack due to reduced</td>
<td>Deformation of subgrade base course</td>
<td>Elongation test</td>
<td></td>
<td>Ductility → Elongation</td>
<td>Bending test</td>
<td>Deformation followability → Bending strain</td>
</tr>
<tr>
<td>Pavement bearing</td>
<td>Bending test</td>
<td>Bending test</td>
<td></td>
<td>Deformation followability → Bending strain</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rutting</td>
<td>Plastic deformation</td>
<td>60°C viscosity test</td>
<td></td>
<td>Viscosity at service temperature → 60°C viscosity</td>
<td>WT test</td>
<td>Plastic deformation resistance → Dynamic stability</td>
</tr>
<tr>
<td></td>
<td>DSR test</td>
<td>Deformation resistance → G*sinδ</td>
<td></td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>
4.2 Advanced modified asphalt

To modify the asphalt, SBS resin and special petroleum resin were used in combination, and those with a high effect for improving SBS resin's drawability of B (butadiene) were used as the process oil.

By using these materials, we thought it would become possible to simultaneously give the "property of suppressing cracking" and the "property of increasing plastic deformation resistance," which are generally said to be in a trade-off relationship. We developed the Advanced modified asphalt (hereinafter, "Development As") that satisfies all performance criteria for the aforementioned evaluation items by repeating trial production multiple times.

4.3 Binder properties test results for Development As

1) Fraass embrittlement point test and bending test results

Table 2 shows the Fraass embrittlement point of Development As and Modified Type II as well as the bending test results together with general properties. As the table shows, Development As has a Fraass embrittlement point that is -27°C lower than Modified Type II. Also, while the bending strain was unmeasurable (a very small value) for Modified Type II, it was 198.1 x 10^{-3} for Development As.

As a result, Development As has excellent deformation followability in the low temperature range, and it can be expected to suppress cracking that occurs due to thermal stress cracks and compressive deformation of the subgrade/base course.

2) DSR test

Table 3 shows the DSR test results for Development As and Modified Type II. In addition, G*sinδ was measured at a temperature of 25°C and an angular velocity of 10 rad/sec, and G*/sinδ at a temperature of 60°C and an angular velocity of 10 rad/sec. As the table shows, in the case of Development As, G*sinδ was 1/16, and G*/sinδ was about 1.17 times that of Modified Type II. It can thus be evaluated that the smaller the value of G*sinδ obtained in the DSR test, the higher the fatigue crack resistance, and the larger the value of G*/sinδ, the higher the plastic deformation resistance.

The results indicated that Development As has high fatigue crack resistance and higher plastic deformation resistance than Modified Type II; it can be expected to suppress the occurrence of fatigue cracks, top-down cracks, reflection cracks, and rutting.

Table 2  Basic properties of Development As

<table>
<thead>
<tr>
<th>Item</th>
<th>Development As</th>
<th>Modified Type II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Penetration (1/10 mm)</td>
<td>114</td>
<td>55</td>
</tr>
<tr>
<td>Softening point (°C)</td>
<td>96.5</td>
<td>61.5</td>
</tr>
<tr>
<td>PI (Penetration Index)</td>
<td>8.85</td>
<td>1.57</td>
</tr>
<tr>
<td>Elongation (15°C) (cm)</td>
<td>92</td>
<td>86</td>
</tr>
<tr>
<td>Elongation (4°C) (cm)</td>
<td>70</td>
<td>54</td>
</tr>
<tr>
<td>60°C Viscosity (Pa · s)</td>
<td>9,610</td>
<td>1,475</td>
</tr>
<tr>
<td>Fraass embrittlement point (°C)</td>
<td>-38</td>
<td>-11</td>
</tr>
<tr>
<td>Bending strain (-20°C) × 10^{-3}</td>
<td>198.1</td>
<td>Unmeasurable</td>
</tr>
</tbody>
</table>

Table 3  Measurement results of G*sinδ and G*/sinδ

<table>
<thead>
<tr>
<th></th>
<th>Development As</th>
<th>Modified Type II</th>
</tr>
</thead>
<tbody>
<tr>
<td>G*sinδ (25°C)</td>
<td>39</td>
<td>642</td>
</tr>
<tr>
<td>G*/sinδ (60°C)</td>
<td>6.56</td>
<td>5.57</td>
</tr>
</tbody>
</table>

3) BBR test

The results of the BBR test are shown in Figure 2. As the figure shows, Development As has an S value about 1/4 that of Modified Type II and an m value about 2 times more. Cracks due to temperature shrinkage at low temperatures can be evaluated by the S value and m value obtained by the BBR test. It can be evaluated that when the S value is small—that is, when the generated stress when shrinking is small—and the m value is large—that is, when the stress relaxation ability at low temperatures is high—the effects of suppressing the generation of low temperature cracks is high.

From this, Development As can be expected to suppress the occurrence of temperature stress cracks and suppress the opening of construction joints.
4.3 Property test results for the mixture using Development As

The properties of the mixture using Development As are described below. The type of mixture was dense graded mixture (13). In the following, the mixture using Development As will be referred to as the "Development mixture," while the mixture using Modified Type II will be referred to as the "Modified Type II mixture." The aggregate synthesis particle size and amount of asphalt used are shown in Table 4.

Table 4  Mixture particle size and asphalt amount

<table>
<thead>
<tr>
<th>Mixture name</th>
<th>Asphalt quantity (%)</th>
<th>Sieve aperture (mm) and passed mass percentage (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dense particle size mixture (13)</td>
<td>5.0</td>
<td>100 96.5 64.4 42.3 26.5 15.5 6.2</td>
</tr>
</tbody>
</table>

(1) Wheel tracking test

The wheel tracking test results are shown in Table 5. From this table, it was confirmed that the dynamic stability of the Development mixture is 6,000 (times/mm) or more and the plastic deformation resistance is applicable to heavy traffic lines.

Table 5  Dynamic stability

<table>
<thead>
<tr>
<th>Mixture type</th>
<th>Developed mixture</th>
<th>Modified type II mixture</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dynamic stability</td>
<td>&gt; 6,000</td>
<td>&gt; 6,000</td>
</tr>
</tbody>
</table>

(2) Bending test

Figure 4 shows the relationship between the test temperature and bending strength in the bending test (2 point supported center 1 point loading method, strain rate of $6.25 \times 10^{-3} (1/S)$), while Figure 5 shows the relationship between the test temperature and fracture strain. This indicates that the embrittlement point of the Development mixture shifts by about $15^\circ$C to the low temperature side, and it has a viscous property in a wide temperature range on the low temperature side compared to the Modified Type II mixture. In addition, Figure 5 shows that the Development mixture's bending strain was about three times larger than that of the Modified Type II mixture.

The results indicate that Development As has excellent deformation followability across a wide temperature range, and it can be expected to suppress cracking that occurs due to compressive deformation and so forth of the subgrade/base course.
(3) Bending fatigue test

Figure 6 shows the cycles to failure for the bending fatigue test. In this research, the test conditions were as shown in Table 6, and the cycles to failure in the test was defined as the inflection point where the stress sharply decreased.

As the figure shows, it was confirmed that the cycles to failure of the Development mixture exceeded 1 million, which was about 60 times larger than that of the Modified Type II mixture, and the crack resistance against bending fatigue was high. Based on this fact, the Development mixture is considered to be resistant to fatigue cracks, top-down cracks, and reflection cracks.

(4) Temperature stress test

The results of the temperature stress test are shown in Figure 7. As the figure shows, the stress relaxation limit point (the limit temperature at which the mixture can relax the stress generated inside) is -12°C for the Modified Type II mixture and -24°C for the Development mixture; thus, it was revealed that the developed mixture has excellent stress relaxation ability.

From this, it can be evaluated that Development As has high resistance to temperature stress cracking and opening of construction joints.

Table 6 Test conditions for the bending fatigue test

<table>
<thead>
<tr>
<th>Item</th>
<th>Setting value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loading method</td>
<td>Both ends fixed 2 point loading</td>
</tr>
<tr>
<td>Test piece size</td>
<td>40 x 40 x 400 mm (span length: 300 mm)</td>
</tr>
<tr>
<td>Test temperature</td>
<td>5°C</td>
</tr>
<tr>
<td>Load condition</td>
<td>Strain control frequency 5 Hz</td>
</tr>
<tr>
<td>Distortion</td>
<td>400 µ</td>
</tr>
</tbody>
</table>
Incidentally, the temperature stress test consists of lowering the temperature of a rod-shaped specimen fixed at both ends as shown in Figure 8 at a constant temperature gradient, which will cause the test specimen to break due to the internal stress which increases as the temperature decreases. This test method imitates the mechanism of temperature stress crack generation. In this test, the temperature gradient was -3°C/hour.

4.5 Summary of the laboratory tests

Based on the above results, Development As can be said to be a modified asphalt that simultaneously possesses properties such as plastic deformation resistance, stress relaxation property, and deformation followability, which are generally in a trade-off relationship. It has been clarified that the crack resistivity of Development As is superior to that of Modified Type II when generally applied to heavy traffic routes.

5. TRIAL CONSTRUCTION

The trial construction was carried out to confirm the Development mixture's workability. Table 7 gives an outline of the trial construction, Table 8 shows the manufacturing and working temperature conditions of the Development mixture at the time of trial construction, and Photo 2 depicts the construction situation.

When producing the Development mixture at the asphalt mixture plant, the mixing time was the same as that of general mixtures, but this did not affect the miscibility. In addition, there was no dragging of the mixture by the asphalt finisher during construction, and the operability of raking performed by human workers was similar to that of ordinary modified asphalt. The degree of compaction by the cut-out core was at least 98%. From these, the Development mixture was judged to have good workability.
Table 7  Conditions of trial construction

<table>
<thead>
<tr>
<th>Item</th>
<th>Setting value, specification, etc.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Blend</td>
<td>Type: Dense particle size mixture (13)</td>
</tr>
<tr>
<td></td>
<td>Asphalt amount: 5.0%</td>
</tr>
<tr>
<td>Construction scale</td>
<td>Width: 3.5 m</td>
</tr>
<tr>
<td></td>
<td>Length: 25 m</td>
</tr>
<tr>
<td></td>
<td>Thickness: 4 cm</td>
</tr>
<tr>
<td>Construction machines</td>
<td>Asphalt finisher: 2.0 to 4.5 m (Vibrator method)</td>
</tr>
<tr>
<td></td>
<td>Rolling machine: Macadam roller (9 t)</td>
</tr>
<tr>
<td></td>
<td>Tire roller (9 t)</td>
</tr>
</tbody>
</table>

Table 8  Quality control for the Development mixture

<table>
<thead>
<tr>
<th>Item</th>
<th>Measured value</th>
<th>Target value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shipment temperature (°C)</td>
<td>180</td>
<td>170 to 180</td>
</tr>
<tr>
<td>Arrival temperature (°C)</td>
<td>178</td>
<td>—</td>
</tr>
<tr>
<td>Spreading temperature (°C)</td>
<td>168</td>
<td>160 or higher</td>
</tr>
<tr>
<td>Initial rolling temperature (°C)</td>
<td>158</td>
<td>140 or higher</td>
</tr>
<tr>
<td>Secondary rolling temperature (°C)</td>
<td>120</td>
<td>100 or higher</td>
</tr>
</tbody>
</table>

6. APPLICATION TO ACTUAL ROAD

A mixture made using Development As was experimentally applied to a single layer cutting overlay work in a highway earthwork. Table 9 gives an outline of the construction. The Cracks penetrating to the base layer occurred in locations where the average traffic volume was about 18,000 vehicles/day; the crack rate was 20% or more.

Generally, construction is done to remove to the base layer and change the surface/base course; however, with an eye to the reflection crack suppression effect of the mixture using Development As, it was applied to the cutting overlay of 4-cm surface course with the intention of shortening the regulation time. For comparison, Modified Type II mixture was applied in the adjoining section.

In addition, there was no dragging of the mixture by the asphalt finisher during construction, and the operability of raking performed by human workers was good; compaction of 98% or higher was obtained. At present, about six months have elapsed since construction, but no reflection cracks have occurred, and the road remains in good condition.

In addition to the above, durability is being confirmed by carrying out test construction in locations with large traffic volumes, such as national roads and roads on the premises of private quarry factories.

Table 9  Condition for the test construction on express highway

<table>
<thead>
<tr>
<th>Item</th>
<th>Setting value, specification, etc.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mixture</td>
<td>Type: Dense particle size mixture (13)</td>
</tr>
<tr>
<td></td>
<td>Asphalt amount: 5.4%</td>
</tr>
<tr>
<td>Construction scale</td>
<td>Width: 4.25 m</td>
</tr>
<tr>
<td></td>
<td>Length: 90 m</td>
</tr>
<tr>
<td></td>
<td>Thickness: 4 cm</td>
</tr>
<tr>
<td>Construction machines</td>
<td>Asphalt finisher: 2.3 to 6 m (Tamper vibration method)</td>
</tr>
<tr>
<td></td>
<td>Rolling machine: Macadam roller (10 t)</td>
</tr>
<tr>
<td></td>
<td>Tire roller (13 t)</td>
</tr>
</tbody>
</table>

7. CONCLUSION

The results of this study are summarized as follows.

i) It was found experimentally that it is possible to diminish the reduction in pavement bearing capacity by suppressing cracking of the asphalt pavement surface course or surface/base course to prevent rainwater from penetrating into the pavement body, which prolongs the pavement service life.

ii) In consideration of the form and factors of cracking, the required performance criteria were sorted out, and modified asphalt was developed in accordance with the results. Thus, by adjusting the type and mixing ratio of the materials used for the modified asphalt, we developed modified asphalt which simultaneously satisfies the performance criteria (e.g., plastic deformation resistance, stress relaxation property, and deformation followability) that can be applied to heavy traffic routes.
iii) By conducting in-house test construction and test construction on an actual road, Development As was confirmed to have mix ability.

8 ACKNOWLEDGEMENT

Authors really express to special acknowledgement to Public Works Research Institute to promote this research effectively.

[References]
1) Pavement Inspection Procedure, Road Bureau, Ministry of Land, Infrastructure, and Transport, October 2016
2) "Kaitai Shinsho" of the asphalt pavement project, Takahashi et al, "Pavement," pp. 13–19 (2016.2)
5) Japan Road Association: Pavement survey and test method handbook, June 2007
KEYWORDS:
Polymer Modified Asphalt, Pavement life, repair and rehabilitation, Durability, Reflection crack

ABSTRACT:
Based on the recently research for a large-scale survey on the express highway in service in Japan, it was essential to prolong the pavement life as following countermeasures, (1) to prevent penetration of rainwater from the road surface, (2) to remain the interlayer adhesive strength between asphalt layers (surface and binder course), (3) to reserve fatigue resistance and the thickness of asphalt base course, and (4) to take measures against stagnation of rainwater in the subbase course.

These means that it is important not to allow rain water to penetrate into the pavement in heavy traffic roadways, or to properly drain the permeated water, thereby maintaining the bearing capacity of the sub-base layer in a sound state which is important for prolonging the life of pavement surface.

Therefore, as one measure to prolong the asphalt pavement life, the advanced polymer modified asphalt was developed not only to improve the rutting resistance, but also to suppress the cracking in the asphalt mixture layer as much as possible. This asphalt mixture is expected to prevent the entry of water into the pavement below the subbase course and to suppress the cracking.

Through this research, the laboratory test and trial construction for the developed modified asphalt mixture were conducted to confirm the effect of the presence of rainwater intrusion into the subbase course.

In this paper, it is reported that the effect of rainwater infiltration on the bearing capacity of the pavement, and also the evaluation on constructability through the trial construction.
Research on Prolonging the Pavement Life with the Advanced Polymer Modified Asphalt

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1 INTRODUCTION

The "Pavement Inspection Procedure"1) established by the Bureau of Public Roads of the Ministry of Land, Infrastructure, and Transport in October 2016 clearly stipulates that pavement inspection shall be done based on three criteria—namely, "crack rate," "rutting amount," and "IRI"—and the target years of use shall be determined to protect the layers below the roadbed in order to prolong the service life. In a report of Takahashi et al., the results of a large-scale open-cut survey on an in-service highway route clarify the damage situation of asphalt pavement during long-term service amidst heavy traffic, including fatigue cracks and permanent deformation, embrittlement due to interlayer adhesion, and roadbed water-bearing. Also, based on these actual damage situations, the following measures to prolong service life are noted to be important: i) countermeasures against penetrating rainwater from the road surface, ii) ensuring interlayer adhesion of the asphalt mixture, iii) ensuring fatigue countermeasures and the layer thickness of the asphalt base (upper layer roadbed), and iv) countermeasures against water bearing in the roadbed.2)

This indicates that, particularly amidst heavy traffic, keeping rainwater and other water from penetrating the pavement body or properly draining the penetrated water in order to soundly maintain the bearing capacity below the roadbed is one important measure for prolonging the service life of asphalt pavement.

Therefore, as a measure to prolong the service life of asphalt pavement, the authors endeavored to suppress cracking in the asphalt mixture layer as much as possible and to prevent water penetration into the pavement body below the roadbed. To this end, we developed modified asphalt, which is excellent in terms of suppressing cracking. More specifically, first, we experimentally verified the influence of rainwater penetration into the roadbed from cracks on the pavement's bearing capacity, and we confirmed the importance of suppressing cracking. In addition, we sorted out a variety of factors related to cracking and the performance required to act as a countermeasure against such factors, and we attempted to develop the advanced modified asphalt that could satisfy the required performance criteria.

In this paper, we report the results of experimental confirmation of the influence of rainwater penetration into the roadbed on pavement bearing capacity as well as the results of the property test of the modified asphalt we developed and the mixture using the advanced modified asphalt, including its evaluation results regarding workability by in-house trial construction and so forth.

2 OVERVIEW OF STUDY

2.1 Confirmation of the influence of water penetration into the pavement body on pavement bearing capacity

At the pavement driving experiment facility of the Civil Engineering Research Institute of the National Research and Development Corporation, i) a road surface section where minor cracks had occurred and ii) a section where minor cracks had been repaired (sealed) with thin overlay cutting were prepared. A loading car was driven on the section for 10 months; thus, the effects of rainwater penetration into the pavement body on the pavement bearing capacity due to the repeated load of the transportation vehicle was verified by the change in the distortion amount by the FWD.

2.2 Development of modified asphalt

The causes of cracking in the asphalt pavement were sorted out by form using the Maintenance Guidebook for Road Pavement 2013 Edition3) and other references. The required performance and evaluation items for the asphalt and asphalt mixture were then set.

During the development of the asphalt, a plurality of modifying agents were selected, and evaluation tests of the binder properties and asphalt mixture properties of prototypes that adjusted the combination and mixing ratio were carried out. Thus, a modified asphalt that satisfied the established required performance criteria was developed.
3. SURVEY IN A LARGE SCALE TEST

3.1 Test method for survey

At the pavement driving experiment facility of the Civil Engineering Research Institute of the National Research and Development Corporation, a test in which a loading car was driven for 10 months on the two adjacent road surfaces described below was carried out.

i) A road surface section in which minor cracks had occurred (hereinafter, the "comparative section")
ii) A section in which minor cracks had been repaired (sealed) with thin overlay cutting (hereinafter, the "rainwater countermeasure section")

The pavement driving experiment facility is a circular circuit with a width of 3.2 m and a length of 628 m upon which an unmanned loading car runs. Photo 1 shows the circumstances of the accelerated load test by the loading car. In the experiment, linear transverse cracks having a width of about 0.2 to 0.4 mm occurred at a crack ratio of approximately 5 to 10%, and the length of each section was set to 16 m (32 m in total).

The loading car was made to run 45,000 times for 10 months (corresponding to 2.7 years with N5 traffic). The amount of distortion by the FWD before and after the loading car was driven were measured, and the influence of rainwater and the repeated load of the transport vehicle on the pavement bearing capacity were confirmed. This verification is part of joint research with the Public Works Research Institute.

3.2 Test results

The rate of increase in the distortion amount after 10 months of driving the loading car compared to the distortion before driving \( \frac{(\text{distortion amount after driving 10 months} - \text{distortion amount before driving})}{\text{pre-running distortion amount}} \times 100 \% \) is shown in Figure 1. The rate of increase in the D0 distortion amount after driving the loading car was 9.0% in the comparative section and 4.3% in the rainwater countermeasure section. The rate of increase in the distortion of D300 was 15.2% for the comparative section and 9.4% for the rainwater countermeasure section. From these results, it can be seen that the rate of increase in both the D0 and D300 distortion amounts of the rainwater countermeasure section are smaller than those of the comparative section, so it was confirmed that reduction in pavement bearing capacity can be diminished by preventing the penetration of rainwater and so forth into the pavement.

From this, it was confirmed that suppressing cracking in the asphalt mixture layer of the surface layer or the surface/base layer is important to maintain the pavement bearing capacity—in other words, to prolong the service life. For reference, the number of days when rainfall of 1 mm or more occurred during the running experiment period of 10 months (February 10, 2015 to December 12, 2015: 305 days) was 94 days, the average rainfall per day on those days was 14.4 mm, and the total rainfall during the experiment period was 1,351 mm.4)
4. DEVELOPMENT OF THE ADVANCED MODIFIED ASPHALT

4.1 Required performance and evaluation items

Regarding the form of cracks that occurred on the asphalt pavement, the required performance and evaluation items for the asphalt and asphalt mixture were sorted out based on occurrence factors. The results are shown in Table 1.

The evaluation tests of the modified asphalt developed based on Table 1 were the (1) Fraass embrittlement point (embrittlement point), (2) bending test (bending strain), (3) DSR test ($G^*\sin\delta$, $G^*/\sin\delta$), and (4) BBR test ($S$ value, $m$ value).

In addition, the tests for confirming the performance of the asphalt mixture were the (1) wheel tracking test (dynamic stability), (2) bending test (embrittlement point, bending strain), (3) bending fatigue test (cycles to failure), and (4) temperature stress test (stress relaxation limit point).

The target performance value was determined to be superior in all evaluation tests to polymer modified Asphalt Type II (hereinafter, "Modified Type II").

Table 1  Required performance and evaluation items for the asphalt and mixture corresponding to the damage forms

<table>
<thead>
<tr>
<th>Damage form</th>
<th>Cause</th>
<th>Main test</th>
<th>Required performance</th>
<th>Index</th>
<th>Test method</th>
<th>Required performance</th>
<th>Index</th>
<th>Test method</th>
<th>Required performance</th>
<th>Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fatigue cracks</td>
<td>Repeated tensile strain on the mixture layer's undersurface</td>
<td>DSR test</td>
<td>Cracking resistance</td>
<td>$G^*\sin\delta$</td>
<td>Bending fatigue test</td>
<td>Fatigue resistance</td>
<td>$S$ value</td>
<td>Cycles to failure</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
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<td></td>
<td></td>
<td></td>
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<tr>
<td>Top-down cracks</td>
<td>Repeated tensile strain on the mixture layer's top surface</td>
<td>DSR test</td>
<td>Cracking resistance</td>
<td>$G^*\sin\delta$</td>
<td>Bending fatigue test</td>
<td>Fatigue resistance</td>
<td>$S$ value</td>
<td>Cycles to failure</td>
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<td></td>
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</tr>
<tr>
<td>Opening of</td>
<td>Accumulation of mixture shrinkage</td>
<td>BBR test</td>
<td>Stress relaxation</td>
<td>$S$ value</td>
<td>Temperature stress test</td>
<td>Stress relaxation limit point</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>construction cracks</td>
<td></td>
<td></td>
<td>property</td>
<td>$m$ value</td>
<td></td>
<td></td>
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<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Temperature stress cracks</td>
<td>Repeated expansion and contraction due to temperature changes</td>
<td>Fraass embrittlement point test</td>
<td>Flexibility at low temperatures</td>
<td>$G^*\sin\delta$</td>
<td>Temperature stress test</td>
<td>Stress relaxation property</td>
<td>$S$ value</td>
<td>Stress relaxation limit point</td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Reflection</td>
<td>Cracking of the construction base</td>
<td>DSR test</td>
<td>Cracking resistance</td>
<td>$G^*\sin\delta$</td>
<td>Bending fatigue test</td>
<td>Fatigue resistance</td>
<td>$S$ value</td>
<td>Cycles to failure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>cracks</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cracks due to</td>
<td>Deformation of subgrade base course</td>
<td>Elongation test</td>
<td>Ductility</td>
<td>Elongation</td>
<td>Bending test</td>
<td>Deformation property</td>
<td>$S$ value</td>
<td>Bending strain</td>
<td></td>
<td></td>
</tr>
<tr>
<td>reduced</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rutting</td>
<td>Plastic deformation</td>
<td></td>
<td>Viscosity at service temperature</td>
<td>$60^\circ\text{C}$</td>
<td>WT test</td>
<td>Plastic deformation property</td>
<td>Dynamic stability</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>
4.2 Advanced modified asphalt

To modify the asphalt, SBS resin and special petroleum resin were used in combination, and those with a high effect for improving SBS resin's drawability of B (butadiene) were used as the process oil.

By using these materials, we thought it would become possible to simultaneously give the "property of suppressing cracking" and the "property of increasing plastic deformation resistance," which are generally said to be in a trade-off relationship. We developed the Advanced modified asphalt (hereinafter, "Development As") that satisfies all performance criteria for the aforementioned evaluation items by repeating trial production multiple times.

4.3 Binder properties test results for Development As

(1) Fraass embrittlement point test and bending test results

Table 2 shows the Fraass embrittlement point of Development As and Modified Type II as well as the bending test results together with general properties. As the table shows, Development As has a Fraass embrittlement point that is -27°C lower than Modified Type II. Also, while the bending strain was unmeasurable (a very small value) for Modified Type II, it was 198.1 x 10^-3 for Development As.

As a result, Development As has excellent deformation followability in the low temperature range, and it can be expected to suppress cracking that occurs due to thermal stress cracks and compressive deformation of the subgrade/base course.

(2) DSR test

Table 3 shows the DSR test results for Development As and Modified Type II. In addition, G*sin\(\delta\) was measured at a temperature of 25°C and an angular velocity of 10 rad/sec, and G*/sin\(\delta\) at a temperature of 60°C and an angular velocity of 10 rad/sec. As the table shows, in the case of Development As, G*sin\(\delta\) was 1/16, and G*/sin\(\delta\) was about 1.17 times that of Modified Type II. It can thus be evaluated that the smaller the value of G*sin\(\delta\) obtained in the DSR test, the higher the fatigue crack resistance, and the larger the value of G*/sin\(\delta\), the higher the plastic deformation resistance.5)

The results indicated that Development As has high fatigue crack resistance and higher plastic deformation resistance than Modified Type II; it can be expected to suppress the occurrence of fatigue cracks, top-down cracks, reflection cracks, and rutting.

Table 2 Basic properties of Development As

<table>
<thead>
<tr>
<th>Item</th>
<th>Development As</th>
<th>Modified Type II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Penetration (1/10 mm)</td>
<td>114</td>
<td>55</td>
</tr>
<tr>
<td>Softening point (^\circ) C</td>
<td>96.5</td>
<td>61.5</td>
</tr>
<tr>
<td>PI (Penetration Index)</td>
<td>8.85</td>
<td>1.57</td>
</tr>
<tr>
<td>Elongation (15(^\circ)C) cm</td>
<td>92</td>
<td>86</td>
</tr>
<tr>
<td>Elongation (4(^\circ)C) cm</td>
<td>70</td>
<td>54</td>
</tr>
<tr>
<td>60(^\circ)C Viscosity (\text{Pa} \cdot \text{s})</td>
<td>9,610</td>
<td>1,475</td>
</tr>
<tr>
<td>Fraass embrittlement point (^\circ) C</td>
<td>-38</td>
<td>-11</td>
</tr>
<tr>
<td>Bending strain (-20(^\circ)C) (\times 10^3)</td>
<td>198.1</td>
<td>Unmeasurable</td>
</tr>
</tbody>
</table>

Table 3 Measurement results of G*sin\(\delta\) and G*/sin\(\delta\)

<table>
<thead>
<tr>
<th></th>
<th>Development As</th>
<th>Modified Type II</th>
</tr>
</thead>
<tbody>
<tr>
<td>G*sin(\delta) (25(^\circ)C)</td>
<td>39</td>
<td>642</td>
</tr>
<tr>
<td>G*/sin(\delta) (60(^\circ)C)</td>
<td>6.56</td>
<td>5.57</td>
</tr>
</tbody>
</table>

(3) BBR test

The results of the BBR test are shown in Figure 2. As the figure shows, Development As has an S value about 1/4 that of Modified Type II and an m value about 2 times more. Cracks due to temperature shrinkage at low temperatures can be evaluated by the S value and m value obtained by the BBR test. It can be evaluated that when the S value is small—that is, when the generated stress when shrinking is small—and the m value is large—that is, when the stress relaxation ability at low temperatures is high—the effects of suppressing the generation of low temperature cracks is high.5)

From this, Development As can be expected to suppress the occurrence of temperature stress cracks and suppress the opening of construction joints.
4.3 Property test results for the mixture using Development As

The properties of the mixture using Development As are described below. The type of mixture was dense graded mixture (13). In the following, the mixture using Development As will be referred to as the "Development mixture," while the mixture using Modified Type II will be referred to as the "Modified Type II mixture." The aggregate synthesis particle size and amount of asphalt used are shown in Table 4.

Table 4 Mixture particle size and asphalt amount

<table>
<thead>
<tr>
<th>Mixture name</th>
<th>Asphalt quantity (%)</th>
<th>Sieve aperture (mm) and passed mass percentage (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dense particle size mixture (13)</td>
<td>5.0</td>
<td>100 96.5 64.4 42.3 26.5 15.5 6.2</td>
</tr>
</tbody>
</table>

(1) Wheel tracking test

The wheel tracking test results are shown in Table 5. From this table, it was confirmed that the dynamic stability of the Development mixture is 6,000 (times/mm) or more and the plastic deformation resistance is applicable to heavy traffic lines.

Table 5 Dynamic stability

<table>
<thead>
<tr>
<th>Mixture type</th>
<th>Developed mixture</th>
<th>Modified type II mixture</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dynamic stability (times/mm)</td>
<td>&gt; 6,000</td>
<td>&gt; 6,000</td>
</tr>
</tbody>
</table>

(2) Bending test

Figure 4 shows the relationship between the test temperature and bending strength in the bending test (2 point supported center 1 point loading method, strain rate of $6.25 \times 10^{-3}$ (1/S)), while Figure 5 shows the relationship between the test temperature and fracture strain. This indicates that the embrittlement point of the Development mixture shifts by about 15°C to the low temperature side, and it has a viscous property in a wide temperature range on the low temperature side compared to the Modified Type II mixture. In addition, Figure 5 shows that the Development mixture's bending strain was about three times larger than that of the Modified Type II mixture.

The results indicate that Development As has excellent deformation followability across a wide temperature range, and it can be expected to suppress cracking that occurs due to compressive deformation and so forth of the subgrade/base course.
(3) Bending fatigue test

Figure 6 shows the cycles to failure for the bending fatigue test. In this research, the test conditions were as shown in Table 6, and the cycles to failure in the test was defined as the inflection point where the stress sharply decreased.

As the figure shows, it was confirmed that the cycles to failure of the Development mixture exceeded 1 million, which was about 60 times larger than that of the Modified Type II mixture, and the crack resistance against bending fatigue was high. Based on this fact, the Development mixture is considered to be resistant to fatigue cracks, top-down cracks, and reflection cracks.

(4) Temperature stress test

The results of the temperature stress test are shown in Figure 7. As the figure shows, the stress relaxation limit point (the limit temperature at which the mixture can relax the stress generated inside) is -12°C for the Modified Type II mixture and -24°C for the Development mixture; thus, it was revealed that the developed mixture has excellent stress relaxation ability.

From this, it can be evaluated that Development As has high resistance to temperature stress cracking and opening of construction joints.

<table>
<thead>
<tr>
<th>Item</th>
<th>Setting value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loading method</td>
<td>Both ends fixed 2 point loading</td>
</tr>
<tr>
<td>Test piece size</td>
<td>$40 \times 40 \times 400 \text{ mm}$ (span length: 300 mm)</td>
</tr>
<tr>
<td>Test temperature</td>
<td>5°C</td>
</tr>
<tr>
<td>Load condition</td>
<td>Strain control frequency 5 Hz</td>
</tr>
<tr>
<td>Distortion</td>
<td>400 $\mu$</td>
</tr>
</tbody>
</table>
Incidentally, the temperature stress test consists of lowering the temperature of a rod-shaped specimen fixed at both ends as shown in Figure 8 at a constant temperature gradient, which will cause the test specimen to break due to the internal stress which increases as the temperature decreases. This test method imitates the mechanism of temperature stress crack generation. In this test, the temperature gradient was -3°C/hour.

4.5 Summary of the laboratory tests

Based on the above results, Development As can be said to be a modified asphalt that simultaneously possesses properties such as plastic deformation resistance, stress relaxation property, and deformation followability, which are generally in a trade-off relationship. It has been clarified that the crack resistivity of Development As is superior to that of Modified Type II when generally applied to heavy traffic routes.

5. TRIAL CONSTRUCTION

The trial construction was carried out to confirm the Development mixture's workability. Table 7 gives an outline of the trial construction, Table 8 shows the manufacturing and working temperature conditions of the Development mixture at the time of trial construction, and Photo 2 depicts the construction situation.

When producing the Development mixture at the asphalt mixture plant, the mixing time was the same as that of general mixtures, but this did not affect the miscibility. In addition, there was no dragging of the mixture by the asphalt finisher during construction, and the operability of raking performed by human workers was similar to that of ordinary modified asphalt. The degree of compaction by the cut-out core was at least 98%. From these, the Development mixture was judged to have good workability.
Table 7  Conditions of trial construction

<table>
<thead>
<tr>
<th>Item</th>
<th>Setting value, specification, etc.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Blend</td>
<td></td>
</tr>
<tr>
<td>Type</td>
<td>Dense particle size mixture (13)</td>
</tr>
<tr>
<td>Asphalt amount</td>
<td>5.0%</td>
</tr>
<tr>
<td>Construction scale</td>
<td></td>
</tr>
<tr>
<td>Width</td>
<td>3.5 m</td>
</tr>
<tr>
<td>Length</td>
<td>25 m</td>
</tr>
<tr>
<td>Thickness</td>
<td>4 cm</td>
</tr>
<tr>
<td>Construction machines</td>
<td></td>
</tr>
<tr>
<td>Asphalt finisher</td>
<td>2.0 to 4.5 m (Vibrator method)</td>
</tr>
<tr>
<td>Rolling machine</td>
<td>Macadam roller (9 t)</td>
</tr>
<tr>
<td></td>
<td>Tire roller (9 t)</td>
</tr>
</tbody>
</table>

Table 8  Quality control for the Development mixture

<table>
<thead>
<tr>
<th>Item</th>
<th>Measured value</th>
<th>Target value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shipment temperature (°C)</td>
<td>180</td>
<td>170 to 180</td>
</tr>
<tr>
<td>Arrival temperature (°C)</td>
<td>178</td>
<td>—</td>
</tr>
<tr>
<td>Spreading temperature (°C)</td>
<td>168</td>
<td>160 or higher</td>
</tr>
<tr>
<td>Initial rolling temperature (°C)</td>
<td>158</td>
<td>140 or higher</td>
</tr>
<tr>
<td>Secondary rolling temperature (°C)</td>
<td>120</td>
<td>100 or higher</td>
</tr>
</tbody>
</table>

6. APPLICATION TO ACTUAL ROAD

A mixture made using Development As was experimentally applied to a single layer cutting overlay work in a highway earthwork. Table 9 gives an outline of the construction. The cracks penetrating to the base layer occurred in locations where the average traffic volume was about 18,000 vehicles/day; the crack rate was 20% or more.

Generally, construction is done to remove to the base layer and change the surface/base course; however, with an eye to the reflection crack suppression effect of the mixture using Development As, it was applied to the cutting overlay of 4-cm surface course with the intention of shortening the regulation time. For comparison, Modified Type II mixture was applied in the adjoining section.

In addition, there was no dragging of the mixture by the asphalt finisher during construction, and the operability of raking performed by human workers was good; compaction of 98% or higher was obtained. At present, about six months have elapsed since construction, but no reflection cracks have occurred, and the road remains in good condition.

In addition to the above, durability is being confirmed by carrying out test construction in locations with large traffic volumes, such as national roads and roads on the premises of private quarry factories.

Table 9  Condition for the test construction on express highway

<table>
<thead>
<tr>
<th>Item</th>
<th>Setting value, specification, etc.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mixture</td>
<td></td>
</tr>
<tr>
<td>Type</td>
<td>Dense particle size mixture (13)</td>
</tr>
<tr>
<td>Asphalt amount</td>
<td>5.4%</td>
</tr>
<tr>
<td>Construction scale</td>
<td></td>
</tr>
<tr>
<td>Width</td>
<td>4.25 m</td>
</tr>
<tr>
<td>Length</td>
<td>90 m</td>
</tr>
<tr>
<td>Thickness</td>
<td>4 cm</td>
</tr>
<tr>
<td>Construction machines</td>
<td></td>
</tr>
<tr>
<td>Asphalt finisher</td>
<td>Macadam roller (10 t)</td>
</tr>
<tr>
<td>Rolling machine</td>
<td>Tire roller (13 t)</td>
</tr>
</tbody>
</table>

7. CONCLUSION

The results of this study are summarized as follows.

i) It was found experimentally that it is possible to diminish the reduction in pavement bearing capacity by suppressing cracking of the asphalt pavement surface course or surface/base course to prevent rainwater from penetrating into the pavement body, which prolongs the pavement service life.

ii) In consideration of the form and factors of cracking, the required performance criteria were sorted out, and modified asphalt was developed in accordance with the results. Thus, by adjusting the type and mixing ratio of the materials used for the modified asphalt, we developed modified asphalt which simultaneously satisfies the performance criteria (e.g., plastic deformation resistance, stress relaxation property, and deformation followability) that can be applied to heavy traffic routes.
iii) By conducting in-house test construction and test construction on an actual road, Development As was confirmed to have mix ability

8 ACKNOWLEDGEMENT

Authors really express to special acknowledgement to Public Works Research Institute to promote this research effectively.

[References]
1) Pavement Inspection Procedure, Road Bureau, Ministry of Land, Infrastructure, and Transport, October 2016
2) "Kaitai Shinsho" of the asphalt pavement project, Takahashi et al, "Pavement," pp. 13–19 (2016.2)
5) Japan Road Association: Pavement survey and test method handbook, June 2007
Mechanistic Performance Prediction of Flexible Pavements at Varying Vehicular Speeds: An Approach to Address Frequency Singularity

**ABSTRACT:** The main objective of this study was to investigate the effect of vehicle speed on fatigue performance of asphalt pavements in contrast to the frequency-singularity design. The study considered three dense graded mixtures and their dynamic complex moduli, $|E^*|$, to understand the impact of varying speeds on fatigue life. The analyses included five levels of speed proportion (-20, -10, 0, +10, and +20% of design speed) of three types of roadways, namely, local, collector, and arterial roads totaling fifteen speed combinations for each mix. $|E^*|$ master curves were developed at 25 °C (77 °F) for the three mixes to predict the dynamic responses at varying loading frequencies. Asphalt mixes with higher moduli showed higher potential of being influenced by change in the speed. In addition, a three-layer elastic system was analyzed using KENPAVE® software to estimate critical tensile strain ($\varepsilon_t$) at the bottom of asphalt layer. $\varepsilon_t$ was found to be higher with decreasing vehicle speeds in comparison with the strain at 10 Hz. Furthermore, sensitivity analysis revealed that an increase in vehicle speed increased fatigue life by about 8-10%, especially for the mix with higher modulus. Thus, it was recommended to incorporate the effect of appropriate loading frequency in the fatigue design of asphalt pavements for more accurate and economical design. Overall, this study provided a sound understanding on the relationships amongst various inputs of fatigue design, and synergized the fundamental understanding of dynamic loading system and the adopted test frequencies within the framework of conventional design practices.
Mechanistic Performance Prediction of Flexible Pavements at Varying Vehicular Speeds: An Approach to Address Frequency Singularity

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1. INTRODUCTION

Fatigue cracking of asphalt mixture employs a complex mechanism of progressive damage caused by repetitive vehicular loading during the service life of flexible pavements. Fatigue design of asphalt pavement encompasses a concurrent consideration of loading characteristics and materials’ resilient responses on the premise of time-temperature relationships. Therefore, a robust design and estimation of fatigue life necessitate a comprehensive understanding of loading characteristics, and its associated effects on asphalt mixtures.

Over the last three decades, the design philosophy of fatigue life has been evolved from an empirical genesis to mechanistic-empirical (M-E) framework. In the M-E approach, the responses of asphalt materials at a defined external loading and temperature conditions are conglomerated by dynamic complex modulus [1]. Currently, $|E^*|$ dynamic complex modulus is used as design input only at one frequency: 10 Hz. This pavement design input parameter based on frequency singularity approach has been considered to reasonably affect the pavement design thicknesses in flexible pavements.

A wealth of literature can be found on the estimation of dynamic modulus, and its dependency on various parameters. A general consensus of research in this field is that the test frequency represents / simulates vehicle speed in real field practice, and influences the dynamic modulus significantly [2-5]. Numerous studies [2-13] investigated the relationships between test frequency and vehicle speed, and explored the empirical, theoretical, and mechanistic spectrum of this concept. Though frequency-vehicle speed relationships provide a sound understanding of the interconversion, the concept is not well-explored in the ambit of design and estimation of fatigue performance. Furthermore, the current design practices [1, 14] idealize the magnitude of modulus at a particular frequency disregarding the dependency of fatigue performance on vehicular speed. Thus, it is deemed important to understand the effect of vehicular speed on fatigue life based on dynamic modulus for two reasons: (a) gain better insights into the sensitivity of various design inputs with respect to vehicle speed, and (b) understand whether the current design practices underestimate / overestimate the fatigue life by employing frequency-singularity design. It is envisioned that this study would synergize the fundamental understanding of dynamic loading systems and adopted test frequencies within the framework of conventional pavement design practices.

Thus, the objective of this study was to investigate the effect of vehicular speed on the fatigue performance of asphalt pavements in contrast to the frequency-singularity design. Furthermore, this study explored the relationships amongst various design inputs pertinent to materials’ responses in respect of vehicular speed. The scope of the effort included (Figure 1):

- Review literature pertaining to vehicle speed-frequency relationships
- Select design speeds and its tolerance ranges for various roadway classifications
- Utilize dynamic modulus database of commonly used asphalt mixtures in order to investigate the effect of frequency on fatigue performance
- Develop dynamic modulus $|E^*|$ master curves of asphalt mixtures, and predict the reduced $|E^*|$ for the frequencies corresponding to the defined vehicle speeds
• Determine the critical strains corresponding to fatigue performance using KENPAVE® software

![Research framework diagram]

*Figure 1. Research framework*

• Estimate the fatigue lives of asphalt mixtures at the derived frequencies, and compare with the fatigue life calculated at the standard frequency, and
• Sensitivity analysis of various design inputs in respect of vehicular speeds.

### 2. BACKGROUND TO VEHICULAR SPEED-FREQUENCY RELATIONSHIP

As mentioned earlier, the responses of asphalt mixtures are measured using dynamic modulus, $|E^*|$ at various temperature-frequency combinations. From an empirical perspective, Yeager [15] suggested that 1, 6, and 12 Hz correspond to 10, 40, and 80 km/h, respectively. Jacobs *et al.* [16] adjudged a loading frequency of 8 Hz equivalent to a vehicle speed of 60 km/h. A study conducted under National Cooperative Highway Research Program (NCHRP), and led by Witczak *et al.* [17] recommended 10 Hz to be used as the loading frequency for highway speed, and 0.1 Hz for intersections and slow-moving traffic. Further, another study conducted at the Asphalt Institute [18] assumes a constant value of 10 Hz regardless of the traffic conditions. In design practices [1, 14] as well, $|E^*|$ at 10 Hz is considered as input. In the mechanistic purview, frequency is computed as a direct inversion of loading time in the US Mechanistic-Empirical Pavement Design Guide (MEPDG) or Pavement ME Design [1]. However, Dongre and coworkers [19] reviewed such direct conversion and could not find any supporting theoretical/mechanistic background. As an alternative to such direct conversion, loading time and cyclic frequency are often converted on the premise of angular frequency as follows:

$$ f = \frac{1}{2\pi t} \quad (1) $$

Where,
Al-Qadi et al. [2] concluded that this conversion should be based on the Fourier analysis of complex viscoelastic function, and suggested correction factors for direct conversion using finite element modeling. Despite disagreement amongst the researchers, Pavement ME Design [2] utilizes the direct conversion between frequency and vehicular speed. Furthermore, the theoretical development of the interconversion between speed and loading time by Brown [20] provided a simplistic relationship using three-layer elastic analysis (Equation 2). This relationship was found to be fairly accurate when thickness of the asphalt layer is in the range of 150-400 mm.

\[
\log(t) = 0.5d + 0.2 - 0.94 \log(v) \tag{2}
\]

Where,
- \( t \) = Loading pulse, s
- \( d \) = Depth of asphalt layer, m
- \( v \) = Vehicle speed, km/h

Similar to the above relationship, a field study conducted using instrumented sections at National Center for Asphalt Technology (NCAT) by Robbins and Timm [21] developed a regression model to estimate strain pulse at the bottom of asphalt layer as follows:

\[
d = j \ln(h) + V^k + T^l + m \tag{3}
\]

Where,
- \( d \) = Strain pulse, s
- \( v \) = Vehicle speed, km/h
- \( h \) = Thickness of asphalt layer, m
- \( t \) = Mid-depth temperature of asphalt layer, °C
- \( k, l, \) and \( m \) = Regression coefficients

It is worth noting that the relationship illustrated in Equation (2) also satisfies the direct conversion conceptualized in Pavement ME Design. Hence, this study utilized Equation (2) as the interconversion tool between vehicle speed and test frequency.

### 3. SELECTION OF INPUT PARAMETERS

#### 3.1 Asphalt mixtures: \(|E^*|\) database

As an initial step towards accomplishing the study objective, three dense-graded asphalt concrete (DGAC) mixtures were chosen from the master mixture database [22] developed at the Arizona State University, USA. The three DGAC (herein designated as “A”, “B”, and “C”) of different binder types and volumetric were chosen to understand the effect of frequency on materials’ design parameters and fatigue performance criterion. Note that three DGAC mixes were selected so as to include a wide range of \(|E^*|\) magnitudes that would enable to conduct a sensitivity study on \(|E^*|\) with respect to vehicular speed. Table 1 summarizes the compositions of asphalt mixtures along with the grade of asphalt binders.
Table 1: Compositions of asphalt mixtures and designations

<table>
<thead>
<tr>
<th>Mix</th>
<th>Place</th>
<th>Air voids, $V_a$ (%)</th>
<th>Effective binder, content, $V_b$ (%)</th>
<th>Asphalt content (%)</th>
<th>Asphalt grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>MnRoad</td>
<td>6.32</td>
<td>11.17</td>
<td>5.50</td>
<td>PG 58-28</td>
</tr>
<tr>
<td>B</td>
<td>NCAT</td>
<td>7.08</td>
<td>8.94</td>
<td>5.04</td>
<td>PG 76-22</td>
</tr>
<tr>
<td>C</td>
<td>Indiana</td>
<td>7.29</td>
<td>8.23</td>
<td>6.63</td>
<td>AC-20</td>
</tr>
</tbody>
</table>

3.2 Design speeds
The design speeds for various classes of roadways were assigned based on the Average Daily Traffic (ADT) of more than 2000 passenger car unit on a rolling terrain [23]. Three roadways, namely local roads, collector, and arterial road were considered with a design speed of 60, 80, and 100 km/h, respectively. Since vehicles on roadway travel with varying speeds, four levels of speed variation, namely -20, -10, 10, and 20% were incorporated along with the base design speed in the analysis. Thus, the analysis included five levels of speed for three types of roadways totaling 15 combinations. All the speed combinations were converted to test frequency using Equation 2 and summarized in Table 2. The standard frequency: 10 Hz, that is conventionally used for fatigue design, was also incorporated in the analysis.

Table 2: Speed spectrum in various types of roadway in USA [23]

<table>
<thead>
<tr>
<th>Roadway Types</th>
<th>Design Speed (km/h)</th>
<th>Variation (%)</th>
<th>Speed (km/h)</th>
<th>Frequency (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Local road</td>
<td>60</td>
<td>-20</td>
<td>48</td>
<td>9.6</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>-10</td>
<td>54</td>
<td>10.7</td>
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<tr>
<td></td>
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<tr>
<td></td>
<td>60</td>
<td>10</td>
<td>66</td>
<td>13.0</td>
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<tr>
<td></td>
<td>60</td>
<td>20</td>
<td>72</td>
<td>14.1</td>
</tr>
<tr>
<td>Collector road</td>
<td>80</td>
<td>-20</td>
<td>64</td>
<td>12.6</td>
</tr>
<tr>
<td></td>
<td>80</td>
<td>-10</td>
<td>72</td>
<td>14.1</td>
</tr>
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<td>80</td>
<td>10</td>
<td>88</td>
<td>17.0</td>
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<td></td>
<td>80</td>
<td>20</td>
<td>96</td>
<td>18.4</td>
</tr>
<tr>
<td>Arterial road</td>
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<td>80</td>
<td>15.5</td>
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<tr>
<td></td>
<td>100</td>
<td>-10</td>
<td>90</td>
<td>17.3</td>
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<tr>
<td></td>
<td>100</td>
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<td>19.1</td>
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<td></td>
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<td>10</td>
<td>110</td>
<td>20.9</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>20</td>
<td>120</td>
<td>22.7</td>
</tr>
<tr>
<td>Standard Frequency</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>10.0</td>
</tr>
</tbody>
</table>

4. RESULTS AND ANALYSES
After determining the frequencies corresponding to the various vehicular speeds, it was essential to estimate the dynamic modulus $|E^*|$ for the evaluation of critical strain, and fatigue life. The following sub-sections document the consecutive steps involved in arriving at the fatigue performance of a selected pavement section at varying vehicular speeds.

4.1 $|E^*|$ determination: master curves

Since the Pavement M-E Design considers $|E^*|$ as a measure of materials’ response parameter, this study addressed the effect of frequency on fatigue life pivoting $|E^*|$ values of the three mixtures at 14, 40, 70, 100, and 130 °F and 25, 10, 5, 1, 0.5, and 0.1 Hz were selected to develop master curves at specific temperature of 77 °F. Note that $|E^*|$ tests were conducted per AASHTO T 378-17 [24]. Table 3 summarizes the $|E^*|$ magnitudes of the three DGAC used in this study.

**Table 3: Dynamic moduli, $|E^*|$ of DGAC at various temperatures and frequencies** [22]

| Temp °F | Frequency Hz | Dynamic Modulus, $|E^*|$ (psi) |
|---------|--------------|-------------------------------|
| 14      | 25           | 5502928 2756856 4903585       |
| 14      | 10           | 5249633 2726343 4812677       |
| 14      | 5            | 5031148 2699382 4735044       |
| 14      | 1            | 4431954 2621302 4521601       |
| 14      | 0.5          | 4136015 2579864 4413936       |
| 14      | 0.1          | 3379333 2461781 4123616       |
| 40      | 25           | 3968319 2352230 3961653       |
| 40      | 10           | 3535822 2252853 3753195       |
| 40      | 5            | 3194467 2168628 3582703       |
| 40      | 1            | 2392282 1942431 3147646       |
| 40      | 0.5          | 2060072 1832320 2945819       |
| 40      | 0.1          | 1370407 1552083 2455154       |
| 70      | 25           | 1589413 1354387 2136277       |
| 70      | 10           | 1233975 1181214 1855225       |
| 70      | 5            | 1001970 1051722 1649064       |
| 70      | 1            | 590053  768851  1207113       |
| 70      | 0.5          | 463198  659304  1037777       |
| 70      | 0.1          | 261340  443221  703551        |
| 100     | 25           | 314061  464151  659406        |
| 100     | 10           | 227074  363363  515156        |
| 100     | 5            | 178736  299482  423731        |
| 100     | 1            | 106373  188108  263483        |
| 100     | 0.5          | 86884  153667  213474         |
| 100     | 0.1          | 57567   97143  130670         |
| 130     | 25           | 64611   145248  153315        |
| 130     | 10           | 52314   111694  116141        |
| 130     | 5            | 45406   92096  94486          |
| 130     | 1            | 34584   60598  59752          |
As the aim of this task was to determine the materials’ responses at calculated frequencies, an attempt was made to predict $|E^*|$ using master curves. Master curve is a tool to illustrate the effects of both temperature and frequency (or time) of the desired viscoelastic parameter ($|E^*|$ in this study) in the linear viscoelastic range. Though there exists a handful amount of shift factor functions and curve fitting models, this study utilized the Arrhenius model with sigmoidal function as followed in the Pavement ME Design [1]. $|E^*|$ data at five temperatures were shifted with respect to frequency until the curves of different temperatures merged into a single smooth function in the form of a sigmoidal shape with respect to a standard reference of 77 °F. Shift functions and curve fitting model are presented as follows:

$$\log[a(T)] = \frac{\Delta E_a}{19.14714} \left[\frac{1}{T} - \frac{1}{T_r}\right]$$

(4)

Where,
- $a(T)$ = Shift factor at temperature, T
- $T_r$ = Reference temperature, °K
- $T$ = Test temperature, °K
- $\Delta E_a$ = Activation energy, curve fitting parameter

And

$$\log |E^*| = \delta + \frac{\alpha}{1+e^{\beta+\gamma(\log f_r)}}$$

(5)

Where,
- $f_r$ = Reduced frequency of loading at the reference temperature, Hz
- $\delta$ = Minimum value of $|E^*|$, MPa
- $\delta+\alpha$ = Maximum value of $|E^*|$, MPa
- $\beta, \gamma$ = Parameters describing the shape of the sigmoidal function

Microsoft Excel® solver function was utilized to minimize the errors between the predicted and observed $|E^*|$ values per Equation 5. After several trials, when the goodness of fit, $R^2$ between predicted and experimentally-observed $|E^*|$ was higher than 0.99, the curve fitting parameters were finalized for developing master curves. Table 4 summarizes the master curve parameters for A, B, and C mixtures at 77 °F.

**Table 4: Master curve parameters for three mixtures at 77 °F**

<table>
<thead>
<tr>
<th>Parameters</th>
<th>A</th>
<th>B</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\log E_{\text{max}}^*$</td>
<td>5.7406</td>
<td>5.4404</td>
<td>5.6905</td>
</tr>
<tr>
<td>$\log E_{\text{min}}^*$</td>
<td>3.1338</td>
<td>3.2496</td>
<td>2.8902</td>
</tr>
<tr>
<td>$\beta$</td>
<td>-0.2076</td>
<td>-0.7910</td>
<td>-1.0162</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>-0.5448</td>
<td>-0.5486</td>
<td>-0.4450</td>
</tr>
<tr>
<td>$R^2$</td>
<td>0.9984</td>
<td>0.9995</td>
<td>0.9998</td>
</tr>
</tbody>
</table>

As the next step, $|E^*|$ was predicted at calculated frequencies (refer Table 2) at 77 °F using master curve parameters (Equation 5). The predicted $|E^*|$ was plotted with respect to the
calculated frequencies to develop master curves for all the mixtures as shown in Figure 2. As well-known, master curves of mixtures provide a preliminary understanding of the comparative response of asphalt mixtures’ performance. For instance, C-mix showed higher time-temperature susceptibility (higher slope, refer Table 4) than the other two mixes indicating that a change in the strain response with respect to frequency for C-mix could be higher than other mixes. $|E^*|$ so predicted were employed to evaluate the critical tensile strain at the bottom of asphalt mixtures as discussed next.

![Figure 2. Master curves for three asphalt mixtures at 77°F](image)

4.3 KENPAVE® analysis: strain estimation
A three-layer elastic layer system was considered to estimate critical strain at the bottom of the asphalt layer. A uniform pavement cross-section was considered for all the mixes, where the modulus of the top layer was the only variable, and characterized by $|E^*|$. A schematic of the pavement cross-section considered for strain estimation is presented in Figure 3. The pavement layers were composed of: subgrade with $E_3 = 50$ MPa under 500 mm of the granular base with $E_2 = 100$ MPa. An HMA layer of 150 mm thickness was assumed on the top of the three-layer system, and $|E^*|$ at various frequencies for all the mixes were utilized as the modulus of this layer.
Since the tensile strain, $\varepsilon_t$ at the bottom of the asphalt layer is critical to the fatigue performance of flexible pavement, $\varepsilon_t$ was numerically estimated using KENPAVE®. To estimate strain, a standard dual axle load set of 80 kN with 724 kPa (105 psi) contact pressure and 93.77 mm contact radius was used to obtain critical tensile strain. Note that a small scale of deviation from the adopted contact pressure would not cause any significant difference in strains due to assumption of the linear elastic behavior of the system. Thus, $\varepsilon_t$ for all the asphalt mixtures were estimated with respect to varying $|E^*|$ at different frequencies as summarized in Table 5.

### 4.3 Fatigue life ($N_f$)

The derived $\varepsilon_t$ was utilized to predict the fatigue life of three asphalt mixtures at varying frequencies using bottom-up fatigue model recommended by Pavement ME Design [1]. The bottom-up fatigue model used for the analysis is as follows:

\[
N_f = 0.00432 \times K_1' \times C \times \left(\frac{1}{\varepsilon_t}\right)^{3.9492} \times \left(\frac{1}{|E^*|}\right)^{1.281}
\]  
\[
K_1' = \frac{1}{0.000398 + 0.003602 \times e^{11.02 - 3.49 \times h_{ac}}}
\]  
(6)

\[
C = 10^M
\]  
(7)

\[
M = 4.84 \left(\frac{V_b}{V_a + V_b} - 0.69\right)
\]  
(8)

Where,

- $N_f$ = Fatigue life, million standard axles, (msa)
- $\varepsilon_t$ = Critical tensile strain, %
- $|E^*|$ = Dynamic modulus at various frequencies, psi
- $K_1'$ = Correction factor for asphalt layer
- $h_{ac}$ = Thickness of asphalt layer, inches
- $C$ = Laboratory-to-field adjustment factor
- $V_b$ = Effective binder content, %
- $V_a$ = Air voids, %
In this process, additional details of the pavement system such as: volumetric of asphalt mix and thickness of the asphalt layer were included as input parameters. Thus, fatigue lives at varying vehicular speeds were predicted in terms of number of repetitions of standard axle load as summarized in Table 5. The variation in fatigue life, $N_f$ with respect to frequency of three mixtures are individually represented in Figure 4. Since the Pavement ME Design employed a frequency singularity of 10 Hz, $N_f$ estimated at other frequencies were compared with fatigue life at 10 Hz (hereafter, designed as $N_f,10$) as marked with yellow horizontal line in Figures 4 (a), (b), and (c). This. As observed, $N_f$ was found to be above the threshold value of $N_f,10$ with increasing frequency indicating that fatigue lives of all the asphalt mixtures could be expected to be higher if the average design speed increases.

With regard to the variation in $N_f$ of individual mixes, a general trend was observed in which an increment in $N_f$ was found in comparison with $N_f,10$. However, the rate of increment in $N_f$ was not consistently accumulated with increasing frequency. This observation was ascribed to the simultaneous dependency of $N_f$ on $\varepsilon_t$ and $|E*|$. The counteracting interaction between the two parameters compounded a nonlinear factor of increment on fatigue life. In effect, lower vehicular speed (or lower frequency) reduced $|E*|$ which consequently increased $\varepsilon_t$, and in turn decreased $N_f$. Therefore, it can be inferred that $N_f$ is sensitive to vehicular speed for all the dense-graded mixtures.

### 4.4 Sensitivity analyses

The sensitivity of design inputs, such as $|E*|$ and $\varepsilon_t$ were measured to quantify the proportional change with respect to vehicular speed. Sensitivity analyses provide insight/clarity to the system variables and their associated effects on $N_f$ in the conventional design concept framework. Figure 5 presents the overall sensitivity of design inputs in the range of -20 to 20% of design speed. Note that the percentage change in the design input values, $\varepsilon_t$, and $|E*|$, for all the three mixes at five different speed levels was estimated with reference to their respective threshold values at 10 Hz (here in referred to as $\varepsilon_t,10$, and $|E*|_{10}$).

As observed in Figure 5(a), $\varepsilon_t$ decreased proportionately with increased speed levels from -20% to +20% for all the mixes. In case of Local road (speed: 60 km/h), the change in $\varepsilon_t$ for the three mixes was marginal with a maximum limit of 3%. On the other hand, $\varepsilon_t$ for all the mixes increased at a higher rate with increasing speed for Collector (speed: 80 km/h) and Arterial (speed: 100 km/h) with a maximum range of ~6 and ~9%, respectively. A comparison amongst $\varepsilon_t$ at various frequencies revealed that the mixes: A, B, and C-mix exhibited a subtle increment in $\varepsilon_t$ by 0.36, 0.38 and 0.46 %, respectively at a frequency of 9.60 Hz in comparison with $\varepsilon_t,10$. Furthermore, $\varepsilon_t$ for all mixes decreased at higher frequencies than 10 Hz indicating that asphalt roadways with higher design speeds would produce a lower critical strain than $\varepsilon_t,10$.

The predicted dynamic responses, which was measured by $|E*|$ showed an increment with increased different speed levels for each of the mixes as shown in Figure 5(b). For all the mixes, $|E*|$ reduced marginally (max: 1%) only when roadways were designed for Local road, and the speed reduced by 20%. In other cases, such as: Collector and Arterial roads, $|E*|$ increased irrespective of change in speed. The maximum increment in $|E*|$ was noticed to be 27, 18, and 16% for A, B, and C-mixes, respectively when the speed increased by 20% in case of Arterial road. Thus, it can be understood that the mix with higher stiffness / modulus (here: C-mix) would result in a higher scale of change in modulus with increasing speed.
Table 5: Estimation of fatigue lives for three asphalt mixtures at various frequencies

| Mixes | $V$ (Km/h) | Frequency, $f$ (Hz) | $|E^*|$ (psi) | Critical tensile strain, $\varepsilon_t$ | $N_f$ (msa) |
|--------|-------------|------------------|--------------|----------------------------------|--------------|
| A      | 48          | 9.60             | 79462        | $5.61 \times 10^4$               | 2.222        |
|        | 54          | 10.73            | 82225        | $5.54 \times 10^4$               | 2.235        |
|        | 60          | 11.84            | 84755        | $5.49 \times 10^4$               | 2.228        |
|        | 66          | 12.95            | 87091        | $5.43 \times 10^4$               | 2.247        |
|        | 72          | 14.06            | 89262        | $5.38 \times 10^4$               | 2.258        |
|        | 64          | 12.58            | 86332        | $5.45 \times 10^4$               | 2.240        |
|        | 72          | 14.06            | 89262        | $5.38 \times 10^4$               | 2.259        |
|        | 80          | 15.52            | 91940        | $5.32 \times 10^4$               | 2.273        |
|        | 88          | 16.98            | 94410        | $5.27 \times 10^4$               | 2.281        |
|        | 96          | 18.42            | 96703        | $5.22 \times 10^4$               | 2.297        |
|        | 80          | 15.52            | 91940        | $5.32 \times 10^4$               | 2.273        |
|        | 90          | 17.34            | 94998        | $5.25 \times 10^4$               | 2.297        |
|        | 100         | 19.14            | 97791        | $5.19 \times 10^4$               | 2.316        |
|        | 110         | 20.94            | 100363       | $5.14 \times 10^4$               | 2.327        |
|        | 120         | 22.72            | 102749       | $5.09 \times 10^4$               | 2.347        |
|        | -           | 10.00            | 80465        | $5.59 \times 10^4$               | 2.218        |
| B      | 48          | 9.60             | 95984        | $5.23 \times 10^4$               | 0.937        |
|        | 54          | 10.73            | 98104        | $5.19 \times 10^4$               | 0.940        |
|        | 60          | 11.84            | 100011       | $5.15 \times 10^4$               | 0.945        |
|        | 66          | 12.95            | 101745       | $5.11 \times 10^4$               | 0.953        |
|        | 72          | 14.06            | 103334       | $5.08 \times 10^4$               | 0.957        |
|        | 64          | 12.58            | 101184       | $5.12 \times 10^4$               | 0.953        |
|        | 72          | 14.06            | 103334       | $5.08 \times 10^4$               | 0.957        |
|        | 80          | 15.52            | 105265       | $5.04 \times 10^4$               | 0.964        |
|        | 88          | 16.98            | 107018       | $5.01 \times 10^4$               | 0.966        |
|        | 96          | 18.42            | 108624       | $4.98 \times 10^4$               | 0.971        |
|        | 80          | 15.52            | 105265       | $5.04 \times 10^4$               | 0.964        |
|        | 90          | 17.34            | 107433       | $5.00 \times 10^4$               | 0.969        |
|        | 100         | 19.14            | 109379       | $4.96 \times 10^4$               | 0.978        |
|        | 110         | 20.94            | 111144       | $4.93 \times 10^4$               | 0.981        |
|        | 120         | 22.72            | 112759       | $4.90 \times 10^4$               | 0.986        |
|        | -           | 10.00            | 96758        | $5.21 \times 10^4$               | 0.942        |
| C      | 48          | 9.60             | 144508       | $4.39 \times 10^4$               | 0.813        |
|        | 54          | 10.73            | 147582       | $4.34 \times 10^4$               | 0.828        |
|        | 60          | 11.84            | 150351       | $4.31 \times 10^4$               | 0.831        |
|        | 66          | 12.95            | 152872       | $4.27 \times 10^4$               | 0.844        |
|        | 72          | 14.06            | 155185       | $4.24 \times 10^4$               | 0.851        |
|        | 64          | 12.58            | 152057       | $4.29 \times 10^4$               | 0.834        |
|        | 72          | 14.06            | 155185       | $4.24 \times 10^4$               | 0.851        |
|        | 80          | 15.52            | 158000       | $4.21 \times 10^4$               | 0.856        |
|        | 88          | 16.98            | 160560       | $4.17 \times 10^4$               | 0.870        |
|        | 96          | 18.42            | 162908       | $4.15 \times 10^4$               | 0.871        |
|        | 80          | 15.52            | 158000       | $4.21 \times 10^4$               | 0.856        |
|        | 90          | 17.34            | 161166       | $4.17 \times 10^4$               | 0.866        |
|        | 100         | 19.14            | 164013       | $4.13 \times 10^4$               | 0.880        |
|        | 110         | 20.94            | 166600       | $4.10 \times 10^4$               | 0.888        |
|        | 120         | 22.72            | 168971       | $4.07 \times 10^4$               | 0.897        |
|        | -           | 10.00            | 145630       | $4.37 \times 10^4$               | 0.820        |
Figure 4. Fatigue lives at various frequencies: (a) A-mix, (b) B-mix, and (c) C-mix
Figure 5. Percentage change in (a) strain, (b) dynamic modulus, and (c) fatigue lives with respect to percentage change in speed.
Since a reverse interaction was found between these two design inputs, $N_f$ was found to be sensitive to change in vehicular speed as shown in Figure 5(c). In case of local road designed with $A$-mix, an increase of 10 and 20 % of vehicular speed increased $N_f$ by 3 and 6 %, respectively. The change in $N_f$ was found to be even higher (8-10 %) for a mix of higher modulus ($C$-mix). On the other hand, a reduction in vehicular speed with respect to the standard design speed was not found to be significant (~1-2%) for all the mixes. In summary, the current fatigue design based on the modulus at 10 Hz underestimates the design life of the asphalt flexible pavements.

5. CONCLUSIONS

The objective of this study was to investigate the effect of vehicular speed on the fatigue performance of asphalt pavements based on current design practice. Three dense-graded asphalt mixes were selected, and the relationships among various design inputs pertinent to materials’ responses in respect of speed were analyzed. The analytical quantification of frequency singularity provided a first of its kind comparative understanding of fatigue design in the conventional design concept. The major findings of the study are summarized as follows:

- Prediction of dynamic responses: with respect to varying speeds was accomplished using master curves at 25 °C (77 °F). A higher time-temperature susceptibility was found for the mix with higher modulus ($C$-mix) that was significantly influenced by the change in vehicular speed.
- Effect of frequency singularity: was analyzed using dynamic modulus, $|E^*|$ for a standard loading on three-layer elastic system. In comparison with standard design speed, critical tensile strain at the bottom of the asphalt layer was found to be higher with decreasing vehicular speeds.
- Sensitivity of fatigue life: was estimated using three base design speeds of major types of roadway. An increase in vehicular speed (-20 to 20 %) increased $N_f$ by 8-10 %, especially for a mix with higher modulus. Hence, the current fatigue design based on the modulus at 10 Hz underestimates the design life of pavement.

It is recommended to incorporate the effect of appropriate loading frequency in the fatigue design of asphalt pavements for more accurate and economical design. Although the study considered a limited number of asphalt mixes, a similar methodology can be utilized to evaluate the frequency singularity on the design life of asphalt mixtures. Further studies are certainly needed in this direction to advance the understanding pertaining to the effect of vehicular speeds on fatigue life of asphalt mixtures that can be correlated to the field data.

REFERENCES


# The Hetauda-Pathlayia Road Pilot Recycling Project: Introduction of FDR, Cement Stabilization and Superpave to Nepal

## Abstract

The MCC Compact with Nepal’s Road Maintenance Project (RMP) aims to deliver improved road maintenance techniques and practices for Nepal’s Strategic Road Network (SRN) in order to reduce overall road agency costs and user costs. A pilot project on the Hetauda-Pathlayia road has been identified for the introduction of several road rehabilitation methods that will help the Department of Roads (DoR) build more comfortable, longer lasting and more structurally appropriate roads, while making almost all use of existing materials.

The first method is Full Depth Reclamation (FDR) with cement stabilization to boost the stiffness of the pavement structure to carry very heavy truck traffic and thereby improve the pavement structure performance over time with lower maintenance costs. The second method is the use of the Superpave asphalt mix design technology along with opportunities for modified asphalts to improve the overall asphalt mix quality namely to resist rutting and thereby reduce overall future maintenance costs. The third method is the recourse to performance based specifications providing bonuses for low roughness (as measured by the International Roughness Index, or IRI) values and penalties for higher values.

While all three methods are well known approaches in many countries, the originality is that they are all thoroughly designed based on the identification of the cause of deterioration, are all integrated in one project, are new to Nepal and are accompanied with a comprehensive technology transfer program. The findings in this paper may therefore have application on heavily trafficked transnational road corridors.
The Hetauda-Pathlayia Road Pilot Recycling Project: Introduction of FDR, Cement Stabilization and Superpave to Nepal

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1 INTRODUCTION

The Millennium Challenge Corporation (MCC) is a small, U.S. Government agency established in 2004 by George W. Bush to fight global poverty in select developing countries with a demonstrated commitment to good governance. MCC provides time limited grant investments to promote economic growth and help people lift themselves out of poverty, creating more stable, secure countries with new business opportunities. The MCC model focuses on advancing economic growth, empowering the poor, modernizing development through the use of data driven analysis with results monitored, measured and publicly shared, catalyzing private sector investment opportunities and promoting shared values like fighting corruption and respecting democratic rights.

In September 2017, MCC and the Government of Nepal (GoN) signed a $500 million Compact comprised of two projects; 1) the Electricity Transmission Project, which aims to increase electricity consumption by facilitating power trade and by improving the availability and reliability of electricity supply in Nepal’s electricity grid; and by facilitating power trade, and 2) the Road Maintenance Project (RMP), which aims to maintain road quality across the strategic road network.

The Millennium Challenge Account-Nepal (MCA-N) is the GoN entity responsible for implementing the Compact. One of the unique aspects of MCC’s Compact structure is that projects have a strict five-year timeline for implementation upon Entry into Force (EIF), which generally takes anywhere from 12-24 months from Compact signature. Before EIF, a pilot project on the Hetauda-Pathlayia road has been identified for the introduction of several methods that will help the Nepal Department of Roads (DoR) build higher quality and more structurally appropriate roads, while making use of existing materials. This results of the pilot road segment will be used to inform the remainder of the $40 million RMP works budget.

2 BACKGROUND

Nepal’s Strategic Road Network (SRN), shown in Figure 1 has tripled in size over the last 20 years from 4,740 km to over 13,000 km today (see Figure 2) in order to provide road access to each regional and provincial capital.
While the focus has been on road network coverage, the problem of maintaining the built road network has become more and more apparent. The Department of Roads (DoR) began collecting IRI data in 2012 on the SRN, as illustrated in Figure 3. To provide a relative comparison with the United States, the US Bureau of Transportation Statistics (BTS, 2018) shows 91% of the measured network below a 3.48m/km level compared to 6% for Nepal less than 4.0m/km. This is consistent with the World Economic Forum Report (Schwab, 2016) for 2016-2017 that ranked the quality of Nepal’s road infrastructure at 118/138 countries as compared to 13/138 for the United States.
As pavements move from “Good” Condition towards “Poor” or “Bad” condition, they become more and more expensive for the Road Agency to bring back to a higher level of service as illustrated in the average square meter costs shown in Figure 4.

Figure 4 shows the various maintenance and rehabilitation intervention stages prevalent in Nepal, with the generally expected continuously decreasing function indicative of the deterioration trend as a result of each intervention.

Over 80% of the Nepal SRN is classified as “Poor” or “Bad,” which translates to an increasingly higher need for costly rehabilitation and reconstruction. Routine and recurrent maintenance expenditures are generally in line with the stated need (i.e. fully funded), however fall short with regards to periodic maintenance interventions. In addition to these road agency costs, the road user is facing ever higher costs due to vehicle operating costs (i.e. wear and tear on the vehicle, increased fuel consumption, etc.), which are modelled using HDM-4 Road User Costs as shown in Table 1.
For example, a heavy goods truck would save approximately $0.18 per kilometer per direction travelled in operating and travel time costs if the IRI were improved from 10m/km to 2m/km.

Additionally, and more tragically, road safety is a major problem in Nepal; the World Health Organization (WHO) in 2015 has estimated approximately 17 fatalities per 100,000 population in Nepal compared to 10.6 in the United States; the WHO estimates that this results in an annual loss of approximately 0.8% of GDP (WHO, 2015). The World Bank notes that the majority of fatalities on the SRN occur outside of the Kathmandu Valley involving mostly trucks and busses with reckless driving as the cause (World Bank, 2013).

3 HETAUDA-PATHLAIYA ROAD

The Hetauda-Pathlaiya Road, shown in yellow in Figure 5 below, is part of the Mahendra Highway, also called the East West Highway, or H1, which runs for just over 1,000 km across the entire width of Nepal through the Terai region. The Hetauda-Pathlaiya segment is a 29.07 kilometer, two lane North to South segment on the H1. It was originally completed in 1982 via cooperation support from the USA and upgraded in 1997 by the Asian Development Bank, with some local resurfacing performed since. The Annual Average Daily Traffic (AADT) is approximately 4,000 vehicles per day (vpd) excluding motorcycles and three-wheelers, of which 44% is considered heavy loads, or trucks. These are mostly overloaded, based on a recent axle load survey showing a Truck Factor, or average number of 8.2 ton Equivalent Single Axle Loads (ESALs), of 4.8, as provided by the Department of Roads (DoR). In another study in the same area, a truck factor of 6.83 (Katahira & Engineers et al, 2015) is reported. The grand majority of trucks are carrying goods from India northbound with most trucks returning empty on the southbound direction. As such, the import volume is approximately 10 times the export volume. The Nepal Traffic Police (NTP) provided the following road fatalities and injury data for the last two years:

<table>
<thead>
<tr>
<th>Period</th>
<th>Reported Deaths</th>
<th>Critical Injury</th>
<th>Minor Injury</th>
</tr>
</thead>
<tbody>
<tr>
<td>2015/2016</td>
<td>10</td>
<td>8</td>
<td>4</td>
</tr>
<tr>
<td>2016/2017</td>
<td>23</td>
<td>18</td>
<td>43</td>
</tr>
</tbody>
</table>

Table 2: Road Fatalities and Injury Data for the Hetauda-Pathlaiya Road, Note that the Nepali Calendar (Nepal Sambat Calendar) is approximately 57 years ahead of the Gregorian calendar.)
The IRI was measured by an ARRB Roughometer Class III device in accordance with ASTM E1926-98 procedures with precision and bias measurements confirmed on four 100 meter calibration sections of varying roughness with errors less than 10%. The heavily trafficked northbound outer wheel path measurements were retained (as this direction had the most traffic), with results presented in Figure 6 below. Note the three various colors represent the homogenous sectioning used in HDM-4.

Considering, that the road is only about 29-km long and will deteriorate further before the scheduled intervention in 2019, the sub-sectioning for the economic analysis was performed on 3 zones, namely one zone of 9 sections with an IRI of 4 (covering IRIs of 5.99 and less), one zone with 7 sections with an IRI of 8 (covering IRIs of 6 to 9.99) and a zone with two sections only with an IRI of 12 for IRIs of 10 and more.

The Nepal Department of Roads (DoR) has experimented with manually placed, non-jointed, Portland Cement Concrete (PCC) on portions of this particular road segment, about 8km south of Hetauda (STA 8+000) towards Pathlayia. PCC is about 10 times stiffer (20,000 to 25,000 Megapascal, or MPa) than conventional asphalt concrete mixes (2,000 to 2,500 MPa at the prevailing equivalent temperature resulting in the same damage in the asphalt as that from all the other temperatures throughout the year). Manually placed PCC requires a high amount of lane closure time to complete as well for removal of the existing pavement, grading, forming, placement, finishing and curing to occur before driving. Secondly, this approach is relatively expensive at about 80 USD/m² and the lack of dowels at the transverse joints will result in eventual faulting and higher roughness values and therefore higher vehicle operating costs.
4 DETERMINATION OF THE CAUSE(S) OF DETERIORATION

Understanding the cause(s) of deterioration resulting in the observed distress is the first step in designing an appropriate engineering intervention; much as a doctor must understand all the possible symptoms in order to arrive at a diagnosis, the engineer needs to thoroughly investigate the proximate cause(s) behind the observed deterioration of the pavement.

A preliminary distress survey assessment of the Hetauda-Pathlaiya road segment in 250 meter increments is shown in Table 3 below, based on the LTPP Distress Identification Manual:

<table>
<thead>
<tr>
<th>Distress</th>
<th>Severity</th>
<th>Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse Cracking</td>
<td>Light</td>
<td>Spalling is observed with minor random cracking</td>
</tr>
<tr>
<td></td>
<td>Severe</td>
<td>Spalling is observed with moderate or higher random cracking</td>
</tr>
<tr>
<td>Longitudinal Cracking</td>
<td>No spalling observed in wheel path</td>
<td>Spalling is observed in wheel path</td>
</tr>
<tr>
<td>Alligator Cracking</td>
<td>No spalling observed with some interconnected cracking</td>
<td>Spalling is observed with moderate to severe interconnected cracking</td>
</tr>
<tr>
<td>Edge Cracking</td>
<td>Less than 25% of length cracked</td>
<td>More than 25% of length cracked</td>
</tr>
<tr>
<td>Peeling</td>
<td>Driver maintains speed</td>
<td>Driver slows down</td>
</tr>
<tr>
<td>Rutting</td>
<td>Less than 10mm</td>
<td>More than 10mm</td>
</tr>
<tr>
<td>Bleeding</td>
<td>Patches of up to 20 x 40cm</td>
<td>Longer stretches than 40cm</td>
</tr>
</tbody>
</table>

Table 3: Distress and Severity levels used for the distress survey assessment

All summary distress are shown in Figure 7 below.
The initial analysis indicated the following: 1) severe rutting (generally between 20-30 mm but up to 60 mm in isolated sections) primarily in the northbound wheel paths, 2) small mesh alligator cracking 3) bleeding, 4) longitudinal cracking and 5) edge cracking. Based on the initial condition and distress assessment, some additional testing was performed in order to determine the most likely cause(s) of deterioration on the road segment. The pavement structure is comprised of a 13cm to 20cm Asphalt Concrete surface course on top of a 10cm, gravelly sand base course (fines content of less than 6%) resting on a 20cm gravelly sand sub-base course.

The asphalt testing indicated a high bitumen content at 7.7% and 9.5% (normal values are between 4% and 6%), which supports the rutting and bleeding observed.

Bleeding occurs when higher than normal asphalt binder content reduces the air voids in the mix design, which are needed to allow the binder to expand under higher temperatures, and move towards the surface.

Rutting can occur as a result of too much binder in the asphalt concrete mix, meaning that the aggregate skeleton is discontinuous because of this excess bitumen. It is noteworthy to mention that the function of the bitumen is to provide cohesion to the mix and the function of the aggregate is to transmit the stresses. If the aggregate skeleton is discontinuous because there is too much bitumen then the asphalt concrete will creep or settle under static or dynamic loading, resulting in rutting. As traffic loading proceeds over time, the asphalt concrete begins to depress around the wheel path as a result of lateral shear failure.

The extent of the rutting is due to a structural failure (which is further supported by the small mesh alligator cracking noted) of the base course under heavy loading due to excessive permanent compressive strain; this is especially noticeable on the north bound side, which is where the grand majority of loaded trucks travel from India through Nepal.

Longitudinal cracking was observed along the centerline, which was most likely due to a construction joint. Edge cracking was observed in selected areas with inadequate and/or non-existent road crowns, longitudinal and
sub-surface drainage. Some minor raveling was also observed in segments, where the asphalt concrete was most likely too cold when placed thus resulting in insufficient compaction.

From a structural perspective, deflections were performed on selected representative locations averaging 38/1000mm (range 12/1000 to 92/1000 mm) for the southbound direction and 26/1000mm (8/1000mm to 66/1000 mm) for the northbound direction.

The conclusion of this preliminary engineering design assessment indicates the following: 1) the need for a more rigid pavement structure (Wearing Course and Base Course) to counter rutting caused by heavy traffic loading; 2) a performance based asphalt mix procedure to reduce rutting and possibly bleeding caused high traffic loading and excessive asphalt content; 3) addition of polymer to the bitumen mix to reduce cracking initiation from oxidation, and 4) addition of fiber to the bitumen mix to increase stiffness against rutting and traffic loading, but also to reduce cracking initiation.

5 ROAD SAFETY AUDIT (iRAP)

A road safety design improvement analysis was undertaken by iRAP (www.irap.org), which rated the existing road segment as a 1 and 2 star road (out of 5 stars, with 5 being the safest). iRAP has provided a detailed strip map of proposed interventions to bring the road segments up to at least three stars, with a preliminary estimate of over 4,000 fatalities and serious injuries that would be prevented over 20 years. The proposed iRAP recommendations will be analyzed and integrated as part of the final engineering design.

6 PROPOSED DESIGN SOLUTION

A. FULL DEPTH RECLAMATION (FDR)

In collaboration with the DoR, MCC has determined that another potential design solution than PCC is feasible for this road segment, which may in turn be more widely applied on the rest of the SRN to improve the overall network condition at much lower costs than the existing rehabilitation/reconstruction costs. Full Depth Reclamation (FDR) with cement stabilization was selected as it offers several key advantages. Those are: 1) a relatively simple execution procedure that can be completed in one day reducing traffic deviation concerns, 2) an environmentally sensitive procedure as existing materials are fully reused, which reduces landfill need and material hauling (diesel trucks) and preserves virgin material for future generations; 3) improves the stiffness of the pavement structure and reduces probability of rutting into the base layer (safety and comfort advantages), and 4) much more cost effective at about 25 to 50% less cost than removal and replacement of the old pavement (Jones et al, 2015) and (PCA, 2010).

The preliminary FDR execution procedure will pulverize and mix the existing asphalt concrete into the base course while adding sufficient cement to reach an approximate 3,000 MPa stiffness; cement will be spread and mixed into the recycled base course, compacted, sprayed with curing sealant and paved one lane at a time keeping the other lane open to traffic.

MCA-Nepal plans to recruit Soils Stabilization Expert to assist the DoR in the development and use of cement stabilization in Nepal, while also developing the engineering design and technical specification requirements including the construction procedure and required QA/QC forms for implementation.

Nepal’s cement industry currently meets about 80% of the annual 5 million tones needed in Nepal, with the remaining coming from India. Nepal is currently meeting 70% of its clinker production needs, with the Cement Manufactures Association for Nepal estimating meeting 100% of the clinker demand by 2019 (New Business Age, 2017). The development of cement stabilization technology in Nepal will not only improve the road stiffness and resistance to structural deformation with lower overall maintenance costs (as compared to the current approach) for the DoR, but also provide an economic benefit to Nepali cement producers while also reducing vehicle operating costs as well.

One of the major constraints identified by the cement industry in expanding production and installed capacity production rates is the unreliable supply of electricity; cement production requires a high degree of continuous electricity supply. MCC’s Compact with Nepal has another project, the Electricity Transmission Project, whose objective is to increase domestic electricity supply for consumption by improving the availability and reliability of electricity in Nepal’s electricity grid.
B. SUPERPAVE AND HIGH MODULUS ASPHALTS

Improving the quality and performance of the asphalt wearing course for the pilot project was a principle consideration as this approach can be more readily adapted to the DoR road maintenance procedures, which is a key component of the Road Maintenance Project. Two innovations for the Nepal context are being proposed for the Asphalt Concrete Wearing Course: 1) the use of Superpave, and 2) the use of high modulus materials like polymers and fibers to reduce overall lifecycle costs for the DoR.

Developed by the Strategic Highway Research Program (SHRP) in 1994, Superpave was designed to determine pavement performance through the interaction of pavement structure, traffic, and environment with the paving mix (Cominsky et al, 1994). One of the major innovations of this approach over the Marshall method is to use a gyratory compactor to simulate traffic loading over time in lieu of a specific hammer count. Application of Superpave to the Nepal context is expected to improve overall wearing course quality and reduce rutting potential through improved formulation and testing.

In addition to Superpave, High Modulus Asphalts will be evaluated to improve the overall service life and reduce lifecycle costs, mainly through reduced maintenance needs. Moghadam et al (2011) have noted that the inclusion of fibers, provide reduced fatigue and rutting properties through a mesh like binding of all aggregates. Distin et al (2014) note that the use of high modulus asphalts has also been shown to provide improved structural life (meaning less structural related maintenance) with lower lifecycle costs. Timm et al (2013) found that high modulus asphalts performed better at higher temperatures with lower rutting performance and improved fatigue life relative to a traditional asphalt mix.

MCA-Nepal plans to recruit a Superpave Asphalt Mix Design and Pavement Rehabilitation Engineer to assist the DoR in acquiring the needed equipment and developing and using Superpave in Nepal, while also developing the engineering design and technical specification requirements including the construction procedure and required QA/QC forms for implementation during the construction phase.

C. IRI PERFORMANCE BASED SPECIFICATION

Lower IRI indicates a smoother pavement, which provides lower vehicle operating cost (VOC) and travel time costs (TTC) for the road user (Chatti et al, 2012), and theoretically lower maintenance costs for the road agency.

Previous experience with IRI performance based specifications have shown that local contractors are willing and able to improve current practices when properly incentivized through performance based specifications (Towles et al, 2017). This same experience will be applied to the pilot project based on simulated HDM-4 analysis of the difference in vehicle operating cost savings from various IRI thresholds.

The average IRI for newly delivered AC pavement is around 2.8m/km in Nepal, however for the Hetauda to Pathlayia road segment, MCA-Nepal plans that the bidding documents will set a target for the mean (inner and outer wheel paths) of 1.20 m/km to receive a payment item bonus (% to be set with HDM-4 analysis, but generally around 10%), with partial payment between 1.21m/km and 1.70 m/km and with 1.70m/km or higher being set as the rejection level for payment (Transports Québec, 2009) of the asphalt surfacing; the final scheme will be set during the finalization of the tender documents with the final pre-works economic assessment.

7 ECONOMIC ANALYSIS

Using the existing DoR HDM-4 Calibration, the road segment was sub-sectioned into three homogenous sections by IRI as previously noted. Distress values were summarized per Figure 7 above.

The DoR provided fleet data and traffic counts from 2016 and field verified by MCC via a windscreen survey to ensure reasonableness. The geometry was set at 300 meters altitude with rolling/moderate vertical alignment and fairly straight horizontal alignment. Routine maintenance was considered when there were more than 10 potholes per km and all edge breaks were addressed as they occurred.

HDM-4 uses the Adjusted Structural Number (SNP), which is a modified version of the original AASHO Structural Number (SN) (Rolt et al., 2000). The SNP tries to measure overall pavement strength by weighting the thicknesses of the different layers by a structural coefficient, function of the stiffness of each layer, and also the subgrade (Morosiuk et al, 2004).
The HDM-4 default value SNAP layer coefficient for a traditional Hot Mix Asphalt (HMA) layer at 30°C varies between 0.30 and 0.45 depending on the modulus of the Asphalt Concrete (AC) layer (between 1,500MPa and 4,000MPa respectively). The Base Courses values vary depending on the type of material used. For the existing pavement structure, or the Do Nothing model (Base), the deflection measurements were input into HDM-4 to determine the SNP.

Regarding the FDR with Superpave (FDR-ACP) model SNP, Reeder et al (2017) recommend using layer coefficients for the FDR cement stabilized base course between 0.20 (for 3,557MPa) and 0.26 (for 5,033MPa) range. The base course value selected for this project was 0.24 based on the compressive strength expected.

For the High Modulus Asphalt coefficient, Peret et al (2004) have found that the use of high modulus bitumen, often referenced by the French acronym “EME,” or “Enrobe a Module Élevé,” have been shown to provide increased stiffness from 6,000MPa at 30°C, with Montanelli (2013) finding values up to 7,300 MPa at 29°C. Qi et al (1995) found high levels of stiffness as well and recommended layer coefficients between 0.71 and 0.74. A value of 0.51 was selected.

A structural verification was performed to check the tensile and compressive strains in the pavement structure with WinJulea. A comparison between the Do Nothing Model and the FDR with Superpave high performing asphalt is shown in Table 4 below along with the assumed modulus for the FDR with Superpave asphalt option used and verified with WinJulea.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Do Nothing Model (Base)</th>
<th>FDR with Superpave</th>
<th>Assumed Modulus for FDR with Superpave</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt Concrete</td>
<td>0.41</td>
<td>0.51</td>
<td>4,000 MPa</td>
</tr>
<tr>
<td>Base Course</td>
<td>0.20</td>
<td>0.24</td>
<td>3,000 MPa</td>
</tr>
<tr>
<td>Subbase</td>
<td>0.07</td>
<td>0.07</td>
<td>300 MPa</td>
</tr>
</tbody>
</table>

Table 4: Adjusted Structural Numbers used in HDM-4 with assumed modulus for each layer used in structural analysis

The calibration coefficients were kept at the default values for cracking initiation and progression for the Do Nothing model (Base), while these were adjusted for the FDR with Superpave (FDR-ACP) Option. A literature review was undertaken to justify cracking initiation and progression calibration for High Modulus Asphalts with or without FDR in HDM-4, however none were uncovered.

Absent empirically verified FDR with Superpave higher performing asphalts for HDM-4, a theoretical model was assumed based on others findings using High Modulus Asphalts. The theoretical model used in HDM-4 for the FDR with Superpave (FDR-ACP) assumed a 40% longer period for structural cracking initiation (Kcia=1.4) with a 40% reduction in structural cracking progression (Kcpa=0.6). This assumption will be verified through post construction assessment work as part of the Road Maintenance Project.

The post project IRI was set at 1.20 m/km, which is part of the IRI performance based specification approach. The HDM-4 predicted value for IRI deterioration output is shown below.
Figure 8: HDM-4 Comparison of the Do Nothing Approach (Base) relative to the proposed FDR with Superpave asphalt (FDR-ACP) with a before project IRI of 8.0 m/km in year 2020.

Preliminary cost estimates based on local materials prices indicate an approximate investment cost of $300,000 per kilometer including construction costs contingencies, engineering and environmental and resettlement costs. These will be refined during the final engineering design phase, including the final road safety interventions and associated savings. The current preliminary estimated economic net savings are shown in Table 5 below.

<table>
<thead>
<tr>
<th>Estimated Net Savings</th>
<th>USD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vehicle Operating Cost Savings (VOCs)</td>
<td>$40 million</td>
</tr>
<tr>
<td>Travel Time Savings (TTC)</td>
<td>$5 million</td>
</tr>
<tr>
<td>Road Agency Savings</td>
<td>$15 million</td>
</tr>
<tr>
<td>Reduced Fatality and Serious Accident Savings</td>
<td>TBD</td>
</tr>
<tr>
<td><strong>Total Estimated Preliminary Net Savings</strong></td>
<td><strong>$60 million</strong></td>
</tr>
</tbody>
</table>

Table 5: Preliminary estimated discounted (10%) economic savings for the 28km road section.

8 CONCLUSION

This paper highlighted the process that was undertaken by MCC with the Nepal DoR to develop a pilot road program to apply many various international approaches to the Nepal context in order to assist the DoR develop more robust and lower cost maintenance and rehabilitation options for the SRN. Lessons from the pilot project will be woven into the remainder of the project design.

Figure 9: Expected outcomes for the Nepal Road Maintenance Project Pilot Recycling Program

The expected outcome of the project are listed below (see Figure 9 above for visual representation):

1. Provide a demonstration project for the DoR to evaluate FDR with cement stabilization as a construction method to reduce financial and environmental costs on the SRN;
2. Provide a demonstration project for the DoR to evaluate the use of High Modulus Asphalts to reduce lifecycle costs (less maintenance) and better resist overloading;
3. Provide a demonstration project for the DoR to evaluate Superpave to improve the overall performance of asphalt concrete in Nepal to reduce rutting (and increased road safety) and provide a higher quality mix design to reduce lifecycle costs; and,
4. Provide a demonstration project for the DoR to evaluate IRI performance improvement clauses for eventual use in other projects to provide higher road agency savings (less maintenance and longer service life) and to road users through lower VOCs.

9 ACKNOWLEDGEMENTS:
The authors with to acknowledge the support provided by the Nepal Ministry of Physical Infrastructure and Transport (MoPIT), including the excellent support from the personnel at the Department of Roads (DoR) and Department of Transport Management, the Road Board Nepal (RBN) and IMC, Wordwide.

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World Bank (2013), Nepal Road Sector Assessment Study, Main Report.

**PAPER TITLE**: The Development of A Safer Road under the Repeated Flooding Area in Nakhonpanom Province, Thailand

**TRACK**

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<thead>
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<th>POSITION</th>
<th>ORGANIZATION</th>
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<td>CHANCHORN Chiranuwad</td>
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**KEYWORDS:**
UCS – Unconfined Compressive Strength, Coefficient of Permeability, Curing, Natural Rubber Latex, Blanket and High Traffic Volumes.

**ABSTRACT:**
Both a high mountainous road in Chiangmai province and a road beside an irrigation canal in Nakhonayok province were constructed in 2013 and 2015, respectively. In addition, a reservoir at Kalasin province was constructed more than 16 years ago. These projects have an influence on designers’ concepts in using of soil stabilization to develop and create a safer road under the repeated flooding area in Nakhonpanom province. In the case of a long curing time, the bearing capacity and permeability of the mixed existing soil at the subgrade level with cement and polymer must be taken into consideration. Furthermore, Unconfined Compressive Strength (UCS) and Coefficient of Permeability are more than 10 ksc and less than $4.33 \times 10^{-7}$ cm/sec, respectively. In this project, the construction of embankment was designed to use the local materials in order to be a blanket of materials, such as the laterite earth in this province because of the cheapest price. In addition, the application of soil cement polymer and natural rubber latex will be taken into account. The special test of UCS value (curing – 28 days) is more than 17.5 ksc. Moreover, Coefficient of Permeability is also lower than $7.5 \times 10^{-9}$ cm/sec, because designers aim to solve neither the erosion of high water pressure problems nor the natural rubber latex’s prices by the buying of the latex from gardeners under the government’s policy. We hope that the concepts will be a prototype model to implement for roads under the high traffic volumes in coming years.
The Development of a Safer Road under the Repeated Flooding Area in Nakhonpanom Province, Thailand

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1 INTRODUCTION

Due to the government policy about the natural rubber products in Thailand, Department of Rural Roads has encouraged this policy, especially the Natural Rubber Latex mixed with soil cement and polymer. The conceptual design is based on the best practice of the soil cement and polymer of the route to a royal project named as a small house project in Doi Dam, Wiang Haeng, Chiangmai province. Before the construction, it took about 5 hours to travel 20 km and it takes about 50 minutes after the construction. Another two projects are the Ditch Lining of Canal for Irrigation in the North Eastern Part of Thailand (Kalasin Province) and the ground improvement of existing ground of the rural road in Nakhon Nayok Province. The first application of soil cement and polymer was this project, constructed by hand 16 years ago. The second, where was located in the Bangkok Metropolitan Regions – BMR (Nakhon Nayok Province), was constructed in the last 2 years. All projects are still serviced and used by the road users, and as a result the agricultural products in the rural area increased drastically.

2 STATE-OF-THE-ART

There are three projects, which are Doi Dam Project, Ditch Lining Project, and the Ground Improvement of Rural Road Project, and are described in the following parts.

2.1 Doi Dam Project

In case of the CBR of the existing ground, it was lower than 2 %. This leads to the use of ground improvement. The technical data showed the Un-soaked and Soaked Condition of the UCS - Unconfined Compressive Strength to be 21.3 ksc and 9.42 ksc, respectively.

During the rainy season in 2016, the road was able to resist flash floods and landslides. In addition, the soils from sliding road were dug by hands of local people and motorists were able to ride within 15 minutes as can be seen in Figure 1.

Figure 1. The Road Maintenance during the Sliding of Back Slope by Local People for the Road Users to use the Road Normally.
2.2 Ditch Lining Project
The Second project is the Ditch Lining of Canal for Irrigation in the North Eastern Part of Thailand (Kalasin Province). This project applied soil cement and polymer was constructed by hand in 2002. Moreover, the permeability concept was considered in order to protect the seepage or leak of water flow as shown in Figures 2 and 3.

(a) After Compaction                     (b) After Short Curing                    (c) After Long Curing
Figure 2.The Curing has an influence on the Strength of materials and the Coefficient of Permeability
Source: Mitchell and E.J. Jack 1966

<table>
<thead>
<tr>
<th>Cement (%)</th>
<th>Condition</th>
<th>UCS (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>L2(NiMC)</td>
<td>10.57</td>
</tr>
<tr>
<td></td>
<td>L2(NiMC)</td>
<td>21.77</td>
</tr>
<tr>
<td>5</td>
<td>L2(NiMC)</td>
<td>21.77</td>
</tr>
<tr>
<td></td>
<td>L2(NiMC)</td>
<td>10.57</td>
</tr>
<tr>
<td>8</td>
<td>L2(NiMC)</td>
<td>21.77</td>
</tr>
<tr>
<td></td>
<td>L2(NiMC)</td>
<td>10.57</td>
</tr>
</tbody>
</table>

Figure 3. The Construction of Ditch Lining of Canal by Soil Cement and Polymer in the North Eastern Part of Thailand (The effectiveness of permeability of materials)

2.3 Ground Improvement of Rural Road Project
The improvement of rural road is beside the irrigation canal. Before the construction, there were potholes and the failure of crown slope, as illustrated in Figure 4.
The polymer’s physical qualities are composed of the VISCOSITY: 1000-2000 Cps, PH - 10.50 – 12.50, SPECIFIC GRAVITY: 0.90- 1.10, and SOLID Content > 7.85 – 8.35%. In addition, 4 items must be certified from Thailand Institute of Scientific and Technological Research (TISTR). Figure 5 shows the Pavement In place Recycling (Soil Cement and Polymer) by the composition of cement (< 4%) and polymer (0.5 liters per sq.m).

During Summer times, these materials can protect the fire from the local farmers, who prepare for planting the new rice corps. They can also protect the growth of grasses during rainy season. Therefore, maintenance costs decrease substantially every year.

3 CASE STUDY FOR IMPLEMENTATION

There are two projects for the ground improvement of road construction sites in this fiscal year (2018). One is the 44.60 km of Road to the Royal Project in Tak Province, which is located near the border between Thailand and Myanmar. Second is the Development of a Safer Road under the Repeated Flooding Area in Nakhonpanom Province, Thailand. Both are different in the design. The first project is the scarified exiting ground, topped with local materials.
for ground improvement. This material provides for the sub-base course. This course can resist the erosion of flood flow from the high mountain. The second project is the applied natural rubber latex, which is mixed with high quality local materials, and followed DRR (Department of Rural Roads) standards. Five cm of these materials are covered by the sub-base, to 450 m length, so as to protect the water in the repeated flooding area. The cost of construction will be compared each other within 2 years (including the maintenance cost). In this paper, designers will propose only the Repeated Flooding Area Project (2ND Project), as described below.

3.1 The Concepts of Road Design at the Repeated Flooding Area Project (2ND Project)

The Cost of Laterite Earth is lower than the Cost of Crushed Rock in this area, thus designers make a decision to use the soil improvement by the application of natural rubber latex and the soil cement and polymer. Designers have several soil tests in the Laboratory to consider that leads to the good qualification of design. The results of these projects will be presented about the Coefficient of Permeability (cm/sec.) and Unconfined Compressive Strength (UCS). The various classified soils show the fine sand and homogenous clay have the same value of the coefficient of permeability, as illustrated in the following table.

Table 1. Coefficient of Permeability of Various Types of Soils

<table>
<thead>
<tr>
<th>Types of Soils</th>
<th>k_v (cm/sec.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clean Gravel</td>
<td>&gt;1.0</td>
</tr>
<tr>
<td>Sand</td>
<td>1.0-1.0^-3</td>
</tr>
<tr>
<td>Fine Sand</td>
<td>1.0^-3-10^-7</td>
</tr>
<tr>
<td>Homogenous Clays</td>
<td>&lt;10^-7</td>
</tr>
</tbody>
</table>

Source: Muni Budhu 2000

There are two laboratory tests: the Permeability Test and Unconfined Compressive Strength Test (UCS). These are described in the next parts.

3.1.1 Permeability Test (ASTM 2434-68 (2006))

3.1.1.1 In case of Soil Cement and Polymer, the average coefficient of permeability (k_v) is 4.33*10^-7 cm/sec. The details of this test are presented in Figure 6.

![Figure 6. Permeability Test of Soil Cement with Polymer](image)

<table>
<thead>
<tr>
<th>Samplings</th>
<th>k_v (cm/sec.)</th>
<th>K20</th>
<th>Kavg</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4.68*10^-7</td>
<td>4.26*10^-7</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>5.17*10^-7</td>
<td>4.17*10^-7</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>5.00*10^-7</td>
<td>4.55*10^-7</td>
<td></td>
</tr>
</tbody>
</table>

As can be seen that Kavg is in the permeability rang of homogenous clays.


3.1.1.2 In case of Soil Cement, Polymer and Natural Rubber Latex, the average coefficient of permeability (k_v) 7.44*10^-9 cm/sec. The details of this test are described in Figure 7.

![Figure 7. Permeability Test of Soil Cement with Polymer and Natural Rubber Latex](image)

<table>
<thead>
<tr>
<th>Samplings</th>
<th>k_v (cm/sec.)</th>
<th>K20</th>
<th>Kavg</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8.47*10^-9</td>
<td>7.70*10^-9</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>7.71*10^-9</td>
<td>7.02*10^-9</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>8.34*10^-9</td>
<td>7.59*10^-9</td>
<td></td>
</tr>
</tbody>
</table>

3.1.2 Unconfined Compressive Strength Test (UCS)
In case of Soil Cement, Polymer and Natural Rubber Latex, the UCS varied with percent of cement at 28 days, with not more than 5 percent of cement. It leads to the Maximum of Load and UCS. The comparison of Coefficient of Permeability and UCS are shown in Table 2 and 3.

Table 2. Results of UCS from Mixing of Standard of Soil, Cement, Polymer and Natural Rubber Latex

<table>
<thead>
<tr>
<th>Percentage of Cement mixed with Soil, Moisture Content and Natural Rubber</th>
<th>Maximum Load (KN)</th>
<th>Unconfined Compressive Strength (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>16.58</td>
<td>2,111.03</td>
</tr>
<tr>
<td>4</td>
<td>17.02</td>
<td>2,167.05</td>
</tr>
<tr>
<td>5</td>
<td>17.29</td>
<td>2,201.43</td>
</tr>
<tr>
<td>6</td>
<td>17.25</td>
<td>2,196.33</td>
</tr>
<tr>
<td>7</td>
<td>17.08</td>
<td>2,174.69</td>
</tr>
</tbody>
</table>

Table 3. Results of Coefficient Permeability from Mixing of Standard of Soil, Cement, Polymer and Natural Rubber Latex

<table>
<thead>
<tr>
<th>Percentage of Cement mixed with Soil, Moisture Content and Natural Rubber</th>
<th>Unconfined Compressive Strength (kPa)</th>
<th>Coefficient of Permeability (cm/second)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>2,201.43</td>
<td>7.02*10^-9</td>
</tr>
<tr>
<td>4</td>
<td>2,167.05</td>
<td>7.59*10^-9</td>
</tr>
<tr>
<td>3</td>
<td>2,111.03</td>
<td>7.70*10^-9</td>
</tr>
<tr>
<td>6</td>
<td>2,196.33</td>
<td>7.57*10^-9</td>
</tr>
<tr>
<td>7</td>
<td>2,174.69</td>
<td>7.56*10^-9</td>
</tr>
</tbody>
</table>

It can be seen that the UCS Strength Values decrease gradually, however, the values of permeability test increase slightly when raising the percentage of cement. Therefore, the 1 cubic meter of job mix design is composed of varies materials. The quantities of Cement, Polymer, Natural Rubber Latex (dry), Natural Rubber Latex (with water) and pure water (9 % by weight of soil) are 120 kg, 1.404 kg, 13 kg, 39 kg, and 216 kg, respectively. In other words, the composition of mix design as proposed in construction: the cement > 5 percent by weight of compacted lateral soil, the natural rubber latex with polymer > 15 percent by weight of the cement, and the adjustment of pure water > 9 percent.

3.2 Flexible Pavement Design
In case of the flexible pavement design, Criteria of design is approximated for traffic volumes < 750 vehicles/day and the percentage of heavy vehicle < 2%. Designers use the criteria of AI – The Asphalt Institute. This is due to the fact that:

3.2.1 30,000 Accumulation of ESAL –Equivalent Single Axle Load.
\[
\text{ESAL design} = \text{ADT} \times \text{Percent of Truck} \times \text{Design Lane (50%)} \times \text{Traffic Growth Rate} \times 365
\]
\[
\text{Mr} = \frac{\sum [(\text{Axle Wheel}, \omega \times \text{Load Equivalent Factors}], f-n]}{365}
\]
\[
\text{Mr} > 10.3 \times \text{CBR}
\]

3.2.2 200 Mpa. - Resilient Modulus of Laterite Earth (Mr) of Existing Ground (Nakhonpanom Province) is controlled of the pavement structures, which are 5 cm of Asphalt Pavement: Wearing Course, 20 cm of Soil Cement, Base Course: 5 cm of Blanket Layer of Soil Cement Plus Polymer and Natural Rubber, Sub Base (50 cm approximate – height of the good quality of Laterite Earth as equal to HWL – High Water Level). Therefore, the specifications of controlled materials are:

3.2.2.1 Coefficient of Permeability of Sub Base < 7.5*10^-9 cm/sec.
3.2.2.2 Unconfined Compressive Strength (UCS) of Sub Base at 7 days > 37.09 ksc
3.2.2.3 Unconfined Compressive Strength (UCS) of Sub Base at 28 days > 45.39 ksc
3.2.2.4 Coefficient of Permeability of Subgrade < 4.33*10^-7 cm/sec
3.2.2.5 Unconfined Compressive Strength (UCS) of Subgrade > 10 ksc
4 DURING CONSTRUCTION

The independent engineers will control the quality of materials and the processes of construction. The standard tests as presented above are controlled in several steps. The contractor must reconstruct new pavement if UCS Values of Subgrade are lower than 10 ksc and the coefficients of permeability test are the same criteria as the mentioned UCS test.

4.1 Simulated Testing of 100% of Layer Blanket – 50 m. and 75 percent of layer blanket – 400 m.

4.1.1 In case of the simulated test – 100% of Layer Blanket (50 m), designers have located the testing area within 50 meters. The drainage pipes are 5 cm in diameter with 5/8 cm holes are placed every 3 m along the geotextile sheets at vertical and horizontal pavement structures. Other drainage pipes, 2.5 cm in diameter, are installed at the shoulder of pavement structures. In addition, the typical cross sections of two testing sections will be illustrated Figures 8 and 9. The whole project length is 11.108 km, as shown in Figure 8, and the typical cross section of 50 m – Simulated Testing, is presented in Figure 9. The first section shows the 50 m of the 100 percent of layer blanket - typical cross section in order to test and check of the permeability and compressive strength, as shown in Figure 10.

![Figure 8. Locations of 4 Testing Sections](image-url)
Figure 9. Installation of Pipe Testing in the First Section - Simulated Testing 50 m (100 percent of layer blanket)

Figure 10. Typical Cross Section of 50 m (100 % Blanket)
4.1.2 **In case of 75 percent of layer blanket – 400 m**, the Second section is the 75 percent of layer blanket (400 m) - typical cross section in order to test and check only the permeability and compressive strength of the subgrade, as described in Figure 11.

![Diagram](image)

**Figure 11. The Second Test Section (75 percent of layer blanket – 400 m)**

4.2 The Construction Processes or Practical Steps at the Repeated Flooding Area Project (2\textsuperscript{ND} Project)

The construction tests are more important for the practical processes. Therefore, designers use and apply the re-testing materials again to qualify the results.

4.2.1 **Modified Compaction Test of Subgrade**, the processes of existing ground - testing are proposed in Figures 12 and 13.
Figure 12. Maximum Dried Density and OMC in the Testing Areas

Figure 13. Results of UCS – 3 days (curing) and Varied Percent of Polymer

From the field area data in the above table and the below Graph, maximum dried density and OMC are 2.30 g/cm³ and 10 %, respectively. Therefore, the average of $q_u$ is 28.966 ksc for 3 days – curing and these values are more than 10 ksc (minimum requirement). The job mix design 1 uses the 3 % polymer of cement by weight and 5% of cement. Job mix design 2 uses the polymer up to 5% and the control 5 % of cement. It can be seen that the cement and polymer have an influence on the UCS of materials (Average of $q_u$ only 3 days curing is 61 ksc). Furthermore, increasing the polymer has a slight effect on the strength of pavement.
Average of $q_u$ only 3 days curing is 70 ksc. Therefore, designers conclude that the mix design is appropriate for practical construction. This is due to the fact that the cost of construction will be important for the Rural Road Department’s budget in the fiscal year.

Equipment and recycling machines are used to scarify the existing ground or subgrade, as shown in Figure 14.

![Figure 14. Scarified Existing Ground to improve the Subgrade Level](image)

The results of these processes, which are unsatisfied in some areas owing to the scarified – recycling machines that must be calibrated again, are presented in Table 4.

**Table 4. UCS Test after the Construction Processes (Curing 14 days)**

<table>
<thead>
<tr>
<th>Sample</th>
<th>Unconfined compressive strength ($q_u$)</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>STA0+650</td>
<td>8.82</td>
<td>U</td>
</tr>
<tr>
<td>3/6</td>
<td>10.75</td>
<td>S</td>
</tr>
<tr>
<td>5/6</td>
<td>24.60</td>
<td>S</td>
</tr>
<tr>
<td>6/6</td>
<td>8.61</td>
<td>U</td>
</tr>
<tr>
<td>STA0+700</td>
<td>7.94</td>
<td>U</td>
</tr>
<tr>
<td>2/6</td>
<td>26.50</td>
<td>S</td>
</tr>
<tr>
<td>3/6</td>
<td>12.02</td>
<td>S</td>
</tr>
<tr>
<td>4/6</td>
<td>11.98</td>
<td>S</td>
</tr>
<tr>
<td>5/6</td>
<td>26.91</td>
<td>S</td>
</tr>
<tr>
<td>STA0+750</td>
<td>8.69</td>
<td>U</td>
</tr>
<tr>
<td>2/6</td>
<td>5.26</td>
<td>U</td>
</tr>
<tr>
<td>3/6</td>
<td>Sampling of damage section</td>
<td>-</td>
</tr>
<tr>
<td>4/6</td>
<td>9.16</td>
<td>U</td>
</tr>
<tr>
<td>5/6</td>
<td>7.01</td>
<td>U</td>
</tr>
<tr>
<td>6/6</td>
<td>7.15</td>
<td>U</td>
</tr>
<tr>
<td>STA0+800</td>
<td>10.63</td>
<td>S</td>
</tr>
<tr>
<td>6/6</td>
<td>12.26</td>
<td>S</td>
</tr>
</tbody>
</table>

From the results (curing 14 days) of above tables, the collected data station from 0+750 to 0+800 are not satisfied owing to the lower than 10 ksc. The contractor must demolish the materials in this area, because they do not meet standards, and rebuild. Designers check and find the problem, which is not homogeneous materials - problems during mixing of cement, polymer and soil by a re-cycling machine. However, designers find this problem by the coring after curing 28 days at this section again. The result of UCS data at 28 days are accepted (> 10 ksc) as presented in Table 5.
Table 5. Results of Testing of UCS at 14 days and 28 days under Unconfined compressive strength ≥ 10 ksc

<table>
<thead>
<tr>
<th>STA0+650</th>
<th>STA0+700</th>
<th>STA0+750</th>
<th>STA0+800</th>
<th>STA0+805</th>
<th>STA0+850</th>
<th>STA0+900</th>
<th>STA0+950</th>
<th>STA1+000</th>
<th>STA1+050</th>
</tr>
</thead>
<tbody>
<tr>
<td>18.08 ksc</td>
<td>26.50 ksc</td>
<td>25.68 ksc</td>
<td>34.24 ksc</td>
<td>14.34 ksc</td>
<td>12.89 ksc</td>
<td>18.02 ksc</td>
<td>17.12 ksc</td>
<td>32.07 ksc</td>
<td>11.24 ksc</td>
</tr>
<tr>
<td>10.75 ksc</td>
<td>12.02 ksc</td>
<td>17.99 ksc</td>
<td>16.09 ksc</td>
<td>15.45 ksc</td>
<td>10.09 ksc</td>
<td>13.21 ksc</td>
<td>12.00 ksc</td>
<td>11.99 ksc</td>
<td></td>
</tr>
<tr>
<td>28.56 ksc</td>
<td>11.98 ksc</td>
<td>37.44 ksc</td>
<td>10.25 ksc</td>
<td>10.53 ksc</td>
<td>13.21 ksc</td>
<td>13.36 ksc</td>
<td>10.31 ksc</td>
<td>15.30 ksc</td>
<td>11.84 ksc</td>
</tr>
<tr>
<td>24.60 ksc</td>
<td>26.91 ksc</td>
<td>37.68 ksc</td>
<td>11.48 ksc</td>
<td>10.63 ksc</td>
<td>13.49 ksc</td>
<td>24.36 ksc</td>
<td>17.15 ksc</td>
<td>15.48 ksc</td>
<td>33.64 ksc</td>
</tr>
</tbody>
</table>

- Results of the First Test (14 days)
- Results of the Second Test (28 days)

4.2.2 UCS and Permeability Test of Blanket of Sub-Base

Particle size distribution analysis, which is taken into consideration, is includes only 10 samples in the construction area. Consistency analysis is performed of the Soil Classification (SC). Moreover, the Atterberg limits are presented in Table 6. For instances, the OMC is 11.9 %, thus Moisture Contents must be controlled and the Standard Compaction Test (Maximum Dried Density) is 2.13 g/cm.

Table 6. Results of Optimum Water Content and Maximum Dried Density

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>$W_{opt}$ (%)</th>
<th>$\gamma_{max}$ (g/cm$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12.800</td>
<td>2.149</td>
</tr>
<tr>
<td>2</td>
<td>11.500</td>
<td>2.169</td>
</tr>
<tr>
<td>3</td>
<td>12.900</td>
<td>2.107</td>
</tr>
<tr>
<td>4</td>
<td>11.750</td>
<td>2.152</td>
</tr>
<tr>
<td>5</td>
<td>11.450</td>
<td>2.138</td>
</tr>
<tr>
<td>6</td>
<td>11.000</td>
<td>2.115</td>
</tr>
<tr>
<td>7</td>
<td>11.450</td>
<td>2.167</td>
</tr>
<tr>
<td>8</td>
<td>11.800</td>
<td>2.148</td>
</tr>
<tr>
<td>9</td>
<td>11.800</td>
<td>2.088</td>
</tr>
<tr>
<td>10</td>
<td>12.300</td>
<td>2.078</td>
</tr>
<tr>
<td>Avg.</td>
<td>11.875</td>
<td>2.131</td>
</tr>
</tbody>
</table>

The Results of Tests during Design and during Construction are presented in Table 7. The construction tests are divided into two concepts, which are mortar and concrete concepts. The Average Unconfined Compressive Strength - UCS at 7 days (Trial 2) for 3 samples is 21.32 ksc and it is more than 17.5 ksc, as presented in Table 8.
Table 7. Comparison of Results of Quantities between during Design and during Construction

<table>
<thead>
<tr>
<th>Materials</th>
<th>Quantities (Design) with water content 9%</th>
<th>Quantities (Construction) Trail 1 with Flow Table Test</th>
<th>Quantities (Construction) Trail 2 with Slump Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Latterite Earth</td>
<td>2,400 kg</td>
<td>2,400 kg</td>
<td>2,400 kg</td>
</tr>
<tr>
<td>Cement (5% wt of Soil)</td>
<td>120 kg</td>
<td>120 kg</td>
<td>120 kg</td>
</tr>
<tr>
<td>Natural Rubber Latex</td>
<td>39 kg</td>
<td>39 kg</td>
<td>39 kg</td>
</tr>
<tr>
<td>Special Polymer</td>
<td>1,404 kg</td>
<td>1,404 kg</td>
<td>1,404 kg</td>
</tr>
<tr>
<td>Pure Water</td>
<td>216 kg (water content 9%)</td>
<td>816 kg (water content 34%)</td>
<td>216 kg (water content 9%)</td>
</tr>
</tbody>
</table>

Table 8. Results of UCS of (Trial 2)

<table>
<thead>
<tr>
<th>Simple</th>
<th>Unconfined compressive strength (ksc)</th>
<th>Average (ksc)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20309</td>
<td>21.32</td>
</tr>
<tr>
<td>2</td>
<td>20024</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>20200</td>
<td></td>
</tr>
</tbody>
</table>

In case of Mortar, the test will use the Flow Table Test. The concrete slump test was considered but is not applicable because moisture content increases from 9% to 34%, leading to a higher flow of material, as can be seen in Figure 15.

In case of Slump, the test results show the best value (21.32 ksc), which is more than 17.5 ksc for Sub Base Structure. Moreover, the Slump Value is about 6-7 inches, which is not more than the criteria of Department of Rural Roads. Permeability Test within 7 days, Coefficient of Permeability is about $7.56 \times 10^{-9}$ cm/sec. Designers accept and expect the values within 28 days to be up to $7.5 \times 10^{-9}$ cm/sec.
Figure 16. Processes of Slump Test

Figure 17 presents the test section of Natural Rubber Latex, Polymer, Cement, Soil and Water. It can be seen that a Pickup Driver drives on the test section after the mixing – 7 days. Furthermore, a Construction Truck – 10 Wheels Driver also drives and stands on the test section area. As a result, there is no critical crack on the pavement and the hair cracks or tension cracks is shown on the top of pavement and it does not effect on the pavement structure.

Figure 17. Test Section of Natural Rubber Latex, Polymer, Cement, Soil and Water

4.2.3 Comparison of Micro Structures of Materials

The Comparison of Micro Structure of Materials are Compacted Soil, Compacted Soil Mixing with Cement and Compacted Soil Mixing with Cement, Modified Polymer and Natural Rubber Latex as illustrated in Figure 18.

Figure 18. Comparison of Three Types of Compacted Soil
There are 3 types of Micro Structures of Materials that are Ca K Values decrease by percent of weight and atomic (Compacted Soil). The Ca K Values slightly increase by percent of weight and atomic (Compacted Soil Mixed with Cement). In case of Compacted Soil Mixed with Cement, Natural Rubber Latex and Modified Polymer, Ca K Values increase drastically by percent of weight and atomic as presented in Figure 19.

![Figure 19. Ca K Values increasing drastically under Compacted Soil Mixed with Cement, Natural Rubber Latex and Modified Polymer.](image)

Frank M. Eaton (1940) suggested that “Potassium is the low member of the series. Na soils adsorbed less moisture than did Ca or Mg soils but more than K soils”. Therefore, the K soil is the least absorbed moisture or the lowest of coefficient of permeability.

5 CONCLUSIONS

This research is the first project so as to design and construct a blanket of embankment in Thailand. Designers aim to find the best practice and the appropriate solution for the type of road construction under the Repeated Flooding Area. Therefore, we try to keep the high water level as equal to the finished grade of blanket level – road. This is due to the fact that this process must be controlled the budget of construction and the road maintenance in the future. To conserve budget and to provide safe road conditions when driving in the rainy season, the Director General of Rural Road Department must allocate the maintenance costs to remedy the suffering of Local People during flooding. In addition, these materials can protect not only the growth of grasses during rainy season but also the fire from the local farmers that destroy the pavement of roads during summer times. Therefore, we hope the concepts of this project will be the prototype for the design and construction of roads under the high traffic volumes in the regional cities and this leads to the encouragement and supporting of the Rubber Gardeners to increase the natural rubber prices in the near future.

REFERENCES

**PAPER TITLE**

Development of UHPC Joint Detail for Florida Slab Beam

**TRACK**

Structural Design, Innovative Materials and Technology

<table>
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**KEYWORDS:**

Accelerated Bridge Construction, Florida Slab Beam, Ultra-high performance concrete

**ABSTRACT:**

The Florida Slab Beam (FSB) is a precast, prestressed, flat-slab beam currently used for short-span bridges (less than about 65 feet) by the Florida Department of Transportation (FDOT). Current FSB design includes a cast-in-place (CIP) concrete deck and joint between adjacent beams. Modified section and joint details were desired by FDOT to expedite construction time. The new section and connection geometry will not require a CIP deck and will utilize ultra-high performance concrete (UHPC) in diamond shaped, female-to-female joints, which will create an ideal section for accelerated bridge construction applications. This study presents preliminary design and analyses that investigate transverse moment capacity using eight finite element models: three 18-inch depth joint models and five 12-inch depth joint models using both regular concrete and UHPC at the connection. A number of different joint details were found to have similar performance to current joint details with CIP decks and joints. An experimental investigation was carried out for small-scale specimens recreating the same geometry configuration and loading scheme. The specimens that show enhanced strength and fatigue capacity at the joint connection will be further analyzed by evaluating the complete longitudinal behavior using large scale specimens.
Development of UHPC Joint Detail for Florida Slab Beam

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1 EXTENDED ABSTRACT

Ultra-high performance concrete (UHPC) is becoming more widely used in bridge construction applications due to its remarkable structural performance. Many departments of transportation have tested and deployed the use of UHPC in bridges around the US. Most of these applications have been to connect precast members (e.g. slabs to beams and slabs, adjacent beams, caps to columns, etc.). The Florida Slab Beam (FSB) is a recently developed section type used for short-span bridges (less than about 65 feet) by the Florida Department of Transportation (FDOT). The FSB system consists of shallow precast, prestressed concrete inverted-tee beams that are placed immediately adjacent to each other and then involve reinforcement and concrete being placed in the inner joints and top deck all in one single cast. A modified design is desired to eliminate the cast-in-place (CIP) deck and allow for UHPC to be used in the joint region, which will allow for accelerated construction.

UHPC is used due to its well-known superior performance, including: high compressive and tensile strength, long-term durability, low permeability, high flowability, and low water-to-cement ratio (all compared to current conventional concrete). The use of UHPC will greatly increase the tensile performance of the joint and has been shown to provide a stronger connection than the slab beam itself according to Graybeal (2010). The construction procedure of the system will be expedited by using UHPC. Slab beams can be first laid down side-by-side longitudinally. Next, backer rods or plates can be placed to seal the bottom-most part of the joint. UHPC can then be mixed and cast on site to fill the joint region and connect adjacent members. Finally, an asphalt overlay could then be used to create the driving surface and take care of any differential camber presented or overfills of the closure pour.

The proposed design and construction method of the slab-span superstructure system intend to enhance resilience and robustness of the superstructure by incorporating UHPC in the longitudinal connection. This will enhance the current design standard by eliminating the concrete deck overlay and providing a reduced reinforcement development length in the closure region with brief traffic delays. Also, the higher compressive and tensile strength of UHPC outperforms typical concrete based on Russell and Graybeal (2013), allowing similar or better diaphragm behavior of the slab-beam system. Different joint details and cross-section geometries were analyzed to determine feasible options for short-span bridge solution. The results from these tasks were used to develop several different cross-section and joint details to investigate further in the small-scale experimental testing focusing on joint transverse capacity behavior and full-scale testing focusing on the overall superstructure behaviour. Results from the small-scale and full-scale experimental testing will be used to finalize the cross-section and joint designs.

2 ACKNOWLEDGEMENTS

Florida Department of Transportation financial support is highly acknowledged.

3 REFERENCES


PAPER TITLE
Bearing Capacity of Cement Concrete Pavement above a Cavity under the Road Surface

TRACK
1.4 Maintenance, Repair, and Replacement Techniques

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KEYWORDS:
Cavity
3D-FEM
PCC-pavement
FWD
Bearing capacity

ABSTRACT:
Cavities under the road surface can be caused by breaks in water and sewerage pipes, insufficient compaction of backfilling soil, and soil liquefaction due to an earthquake. In Japan, many water and sewage pipes are deteriorating due to old age or long use, bringing about an increase in the number of cavities under the road surface. Moreover, in areas affected by soil liquefaction due to the Great East Japan Earthquake, the number of cavities under roads was reported to be 10 times the usual figure.

In this study, aiming to develop the evaluation method for the risk associated with cavities under the road, the bearing capacity of portland cement concrete pavement above a cavity in the base course was analyzed by the three-dimensional finite element method (3D-FEM). It was found that the cavity increases the wheel load stress at the bottom of the cement concrete slab and the increase in stress becomes larger with increasing cavity size or shallower cavity depth. A test pavement simulating portland cement concrete pavement above a cavity was constructed and the pavement response was examined by static loading test and falling weight deflectometer (FWD) test. The surface deflection of the pavement above a cavity was larger than that of pavement with no cavity, especially at the edges and corners of the cement concrete slab. Based on the 3D-FEM analysis results and the experimental results, a method was developed for evaluating the risk associated with cavities under concrete pavement using FWD deflections. (246words)
Bearing Capacity of Cement Concrete Pavement above a Cavity under the Road Surface

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1 INTRODUCTION

Recently in Japan, the road infrastructures that were constructed for the Tokyo Olympics in 1960’s are rapidly deteriorated. The proportion of road infrastructures, which passes more than 50 years after construction, will be increased in the next 20 years. Therefore it is required to be maintained and updated in strategically.

Therefore, Ministry of Land, Infrastructure Transport and Tourism (MLIT) positioned the year 2013 as “The First year of maintenance for social infrastructures”, then had begun the efforts for deterioration of road infrastructure. In order to strategically maintenance and updating, it is need to introduce the concept of the life-cycle cost including not only construction (initial) costs, but also the long-term cost such as maintenance and repairing costs. And these must contribute to enhance the cost-effectiveness of social capital investment.

In Japan, in terms of lowering initial cost and opening to traffic earlier, primarily the asphalt concrete pavement primarily has been applied for road construction than cement concrete pavement. At present asphalt concrete pavement occupied 95% of total highway, on the other hand cement concrete pavement is only 5%.

However, according to a recent survey, cement concrete pavement, at the time of the course of about 25 years after-service, it is clear that is the life cycle cost is 20% lower compared to the asphalt concrete pavement. Since this analysis, cement concrete pavement had be spread by MLIT.

Although the cement concrete pavement behaves a highly durable than asphalt concrete pavement, it takes a lot of time and costs to repair. It is important to conduct preventive maintenance before the damage becomes serious.

In Japan, the cavity that occurs under the road surface is one of the cause of cement concrete pavement would deteriorate. The cavity (under the road surface) will be caused by such as the damage of water supply pipe or sewage pipe buried in the road, the less compaction of backfilling soil, or the liquefaction of the ground caused by an earthquake. Particular in urban areas, the number of cavity caused by damage of water supply pipe or sewage pipe buried in the road has been increasing.

Furthermore, it is reported that 10 times more than annual usual number of cavities was occurred by liquefaction in the areas affected by the Great East Japan Earthquake in 2011. Generally, although the survey on cavity under the road surface is carried out with a Ground Penetrating Rader (hereinafter GPR), in the case of applying on cement concrete pavement, since the presence of reinforcing bars or steel network or the like, it is difficult to detect a cavity.

In addition, GPR will find only the position and depth of the cavity, the severity of the cavity such as impact on the bearing capacity of the pavement cannot be clear.

The authors, in the prior study, have analyzed the effect of the cavity under the cement concrete pavement on the durability with 3DFEM.

As a result, it was found that the pavement life in service would be reduced up to 40% by a presence of cavity.
From these results, it can be said that searching a cavity such as impact on the bearing capacity of the pavement at an early stage results in prolong the service life of cement concrete pavement and leads to a reduction in life-cycle cost.

2 APPROACH

Generally in Japan, GPR is used to search the cavity under road surface. Although in this method, the cavity under road surface could be detected by the image showing the change in the measured waveform, it is difficult to search on cement concrete pavement comparing with asphalt concrete pavement, because of metal in pavement such as tie-bar, dowel bar, wire mesh.

By previous studies, since the bearing capacity of the cement concrete pavement occurring cavity under surface was found to be reduced. It may be possible to determine the presence or absence of the cavity by the bearing capacity measuring device such as FWD.

In this study, the surface deflection of cement concrete pavement with loading under a presence of cavity or not were calculated with 3D-FEM, then the possibility of searching cavity by FWD was examined. Based on 3D-FEM calculation, FWD measurement was conducted on cement concrete pavement field retaining dummy cavity as full scale test to develop the cavity searching method by FWD.

3 3D-FEM ANALYSIS

3.1 Model and Parameters for 3D-FEM Analysis

Analysing model is selected to meet the Japanese typical cross section under design CBR=6 and design traffic volume is less 250 (vehicles/day). The cross section in detail is as follows,

- Pavement types: cement concrete pavement
- Concrete slab size: 5m(Length) × 3.5m(Width)
- Cross section of pavement:
  - Surface course is Cement concrete slab (t=200mm)
  - Base course is Mechanically stabilized crushed stone (t=200mm)

The size of the cavity is 1m × 1m × 100mm (thickness), its position are 4 kinds such as free edge, corner, the center of slab, and joint (Figure.1). And the depth of the cavity are 2 kinds such as roadbed top and bottom of base course (Figure.2).

The wheel load is 98 kN, and loading area is equal to loading plate of FWD(265mm × 256 mm). The wheel load is arranged so as to be directly above the cavity (Figure.1).

The response to loading on normal pavement (means no cavity) and on 8 kinds damaged pavements (means retaining cavity) were calculated by 3DFEM. The elastic modulus and Poisson's ratio of each layer for calculation are shown in Table 1.

<table>
<thead>
<tr>
<th>Item</th>
<th>Elastic Modulus [Mpa]</th>
<th>Poisson's ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement Concrete</td>
<td>30,000</td>
<td>0.20</td>
</tr>
<tr>
<td>Base course</td>
<td>490</td>
<td>0.35</td>
</tr>
<tr>
<td>Sub ground</td>
<td>60</td>
<td>0.35</td>
</tr>
<tr>
<td>Cavity</td>
<td>0</td>
<td>0.00</td>
</tr>
</tbody>
</table>
3.2 Result of 3D-FEM Analysis

The deflection at loading point calculated by 3DFEM, and the value to which it was divided by the deflection of normal pavement (hereinafter, deflection ratio) shown in Table 2 and Figure 3. Although the increase in deflection is small at the free edge and the center of slab, it was observed an increase in deflection more than 10% at the joint and the corner, even if the cavity occurred under the concrete slab. Considering the coefficient of variation of the bending strength of the cement concrete for pavement is 10% or less (2), in the case of the deflection ratio is greater than 1.1, it seems that the cavity would be detected.

Comparing deflection ratio through this assumption, the cavities in the corners or joints could be detected by deflection, meanwhile the cavities in the free edge or the center of slab are difficult to be detected.

<table>
<thead>
<tr>
<th></th>
<th>Amount of deflection(mm)</th>
<th>Deflection Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No Cavity</td>
<td>Top of Base</td>
</tr>
<tr>
<td>Free edge</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Corner</td>
<td>1.19</td>
<td>1.34</td>
</tr>
<tr>
<td>Joint</td>
<td>0.63</td>
<td>0.69</td>
</tr>
<tr>
<td>Center</td>
<td>0.46</td>
<td>0.48</td>
</tr>
</tbody>
</table>
4 FULL-SCALE MEASUREMENT IN THE TEST FIELD

In order to verify the validity of analysis by 3D-FEM, the full-scale cement concrete pavement field retaining cavity under the surface were constructed and deflection was measured by FWD. From this results, the estimation for the presence or absence of the cavity by deflection ratio were discussed.

4.1 Outline of Test Field Measurement

The cement concrete pavement field of 30m (length) and 3.5m(width), was constructed at Taisei Rotec Corporation Institute of Technology as shown in Figure.3. The cross section was consisted of cement concrete slab (t=200mm) laid on base course of mechanically stabilized crushed stone (t=200mm).

In this study, since it is difficult to construct an artificially cavity in the pavement, the hard polystyrene foam was embedded as a simulated cavity in the base course. The size of the simulated cavity (1.0m(W)×1.0m(L)×0.1m(T)) was the same as analysis of 3DFEM.

The cavities were located in 4 different positions such as the free edge, corners, joints, and the centre, and embedded at the top or the bottom of base course. These conditions were the same conditions as 3DFEM. Since loading radius of FWD was 150mm and loading weight of FWD was 97-100kN, the deflection measured by FWD was corrected into 98kN as the reference load by equation (1).

\[ w' = w \times \frac{98.0}{P} \]  

Here, 
\( w \) : deflection measured by FWD (mm)  
\( w' \) : corrected deflection (mm)  
\( P \) : loading weight (kN)

Figure.4 Location of simulating cavity
4.2 Result of Full-Scale Measurement

FWD measuring as detecting cavity was conducted on 11 March, 2016. That day was in the rain, air temperature was 3 - 10 °C, the road surface temperature was 9-13 °C.

The warpage due to temperature was not appeared since the temperature gradients within the cement concrete pavement is 0.1 °C / cm. The measurement result by FWD is shown in Table 3 and Figure 5.

In the case of the presence of cavity at the top or bottom of base course, the deflection ratio at center of slab becomes 1.1 or less. On the other hand the deflection ratios become 1.1 or more at free edge, corner, and joints. Therefore it is considered to be able to determine the presence or absence of the cavity by FWD.

Deflection ratio of simulated cavity is larger than the results of analysis by 3DFEM. The reason for this, in the analysis by 3DFEM, it was regarded as the same elastic modulus, even if material surrounding the cavity. Meanwhile in the actual pavement, occurring loose in material surrounding the cavity makes elastic modulus lowered.

A similar behavior was also reported 3) in the case of asphalt pavement. Although it was difficult to detect the cavity on the free edge by 3D-FEM, the deflection measured on the road in service would tend to larger than 3D-FEM calculation. It will be possible to detect the cavity on the road in service even if on the free edge.

<table>
<thead>
<tr>
<th>Amount of deflection(mm)</th>
<th>Deflection Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No Cavity</td>
</tr>
<tr>
<td>Free edge</td>
<td>0.78</td>
</tr>
<tr>
<td>Corner</td>
<td>0.88</td>
</tr>
<tr>
<td>Joint</td>
<td>0.58</td>
</tr>
<tr>
<td>Center</td>
<td>0.30</td>
</tr>
</tbody>
</table>

Figure.5 The Results of FWD measuring

5 DEVELOP THE Method TO Detect THE Cavity Under THE SURFACE ROAD IN SERVICE

Since the possibility of detecting a cavity in the corner or the free edge is indicated by the deflection measured by FWD, assuming an investigation on the real roadway in service, the trial measurement was conducted on the test field at 1.0m pitch on a line as shown in Figure.6. Then the relationship between deflection and cavity was analysed.
5.1 Trial measurement on Test Field

The trial measurement was carried out on 11 June, 2016. That day was in cloudy, air temperature was 29 - 33 °C, the road surface temperature was 33 - 41 °C. The warpage due to temperature was not appeared since the temperature gradients within the cement concrete pavement is 0.1 °C/cm.

5.2 Result of Trial measurement

The relationship between the deflection and the cavity position in the corner is shown in Figure 7. At the corner retaining cavity, the cavities could be detected by FWD measurement since the deflection is increased greatly as compared to the deflection at the corner with no cavity.

Figure 8 shows the relationship between the deflection and the cavity position at the free edge. The deflection of cement concrete pavement behaves larger around the joint, and smaller away from the joint. Therefore the profile of deflection at the free edge behaves concave as shown A and B in Figure 8. On the contrary, if there is cavity at the free edge, the profile of deflection behaves convex as shown C and D in the figure.8. The peak will appear on the cavity position. Thus it is considered that the presence and position of cavities at the free edges can decided by the shapes of deflection profile.

In addition, the statistical estimation was carried out to detect a cavity by the deflection. The average of deflection at corner retaining no cavity was 810mm and its standard deviation was 24mm. At the confidence level of 99%, the deflection of the upper limit will be 872mm. In this trial measurement, both of deflections at the two corners retaining cavity become larger than 1000 mm., it was found that cavity at the corner can be detected by FWD. In the case of the free edge, the average of the deflection with no cavity was 679mm and its standard deviation was 65 mm. At the confidence level of 99%, the deflection of the upper limit will be 872mm. In this trial measurement, the deflection at free edge retaining cavity at bottom of base course is 851mm(C in Figure 8), at the top of base course is 769mm(D in Figure 8). It was found that at the location of free edge, the cavity at the bottom of base course can be detected by FWD. In this study, the presence of many simulating cavities embedded in the narrow test field was influenced on the average and standard deviation of deflection to be larger.
It is quite a rare case in the real road in service that the cavities are concentrated in narrow areas as this study. It is considered the more likely that cavity in the free edge could be detected, since the variation of deflection will tend to small with retaining no cavity on the real road in service.

**Figure.8 Comparing the result of trial measurement at the free edge**

6 CONCLUSIONS

In this study, the surface deflection of cement concrete pavement with loading under a presence of cavity or not were calculated with 3D-FEM, and the possibility of searching cavity by FWD was examined.

Based on the calculation, FWD measurement was conducted on cement concrete pavement field retaining simulating cavity as full scale test to develop the cavity searching method by FWD. The obtained conclusions of this study are as follows.

(1) 3D FEM Calculation.
The increase of deflection at the free edge or center is small even if retaining cavity, meanwhile at the joint or corner is increased more than 10% (deflection ratio is greater more than 1.1). It seems that the cavity would be detected by deflection at the joint or corner.

(2) FWD measurement on the full-scale cement concrete pavement test field.
In the case of the presence of cavity at the top or bottom of base course, the deflection ratio at center of slab becomes 1.1 or less. On the other hand the deflection ratios become 1.1 or more at free edge, corner, and joints. Therefore it is considered to be able to determine the presence or absence of the cavity by FWD.

(3) Develop the method to detect cavity on the road in service
At the corner retaining cavity, the cavities could be detected by FWD measurement since the deflection is increased greatly as compared to the deflection at the corner with no cavity. And also it is considered that the presence and position of cavities at the free edges can decided by the shapes of deflection profile.

7 ACKNOWLEDGEMENTS

I wish to thanks for Dr. NISHIZAWA for advice on 3DFEM. Furthermore, supports from Taiheiyo Cement Corporation and Taisei Roetec Corporation are gratefully acknowledged.

At the end of the main text you can include a statement of acknowledgement of assistance. Grant or award numbers can be quoted as can departmental publication numbers. Acknowledgements must be brief and confined to persons and organisations who have made significant contributions. Do not include the title or rank of people.

8 REFERENCES


ABSTRACT:

The use of Pervious Concrete Pavement (PCP) is a novel roadway technology application, as it allows water to pass through its porous matrix structure, and significantly contributes to reduction of flash flooding and enhances the recharge of groundwater. In this study, multiple test sections of 150 mm top pervious concrete (PC) layer and 200 mm granular sub-base were constructed, and laid on a compacted natural soil subgrade. The main objective of this research study was to estimate infiltration rate of PC, which was further used to compute the infiltration capacity of the PCP system. The Horton’s equation, which follows an exponential decay model, was used to estimate the infiltration rates for PCP in the field. Permeability of the PCP system was evaluated by falling head method. One of the major tasks of this study was to obtain a range of infiltration rates for PCP systems, which was observed to be between 0.05 and 0.29 cm/s. This pilot study is envisioned to help develop reliable strategies to evaluate hydraulic parameters of PCP materials. Furthermore, this study will advance the current state-of-the-art pertaining to green and sustainable pavement infrastructure systems.
An Innovative Approach to Estimate Infiltration Rate of Pervious Concrete Pavements

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1. INTRODUCTION

One of the potential impacts of urbanization and increased construction of impervious road network on natural ground is the change in the urban hydrological characteristics. These impervious pavement systems restrict the infiltration of rain water into natural ground and increase surface runoff, which ultimately lead to urban flooding [1]. The augmentation of paved surfaces prevents replenishment of groundwater table during and after precipitation. The vehicles moving over pavements discharge pollutants through emissions, which are further accumulated over the impervious pavement surface. Also, surface runoff generated due to rainfall gets combined with the rotten leaves, dirt, and dust present on the pavement surface. These pollutants are then carried away to the receiving water body through underground water-wastewater networks at a rapid rate and contribute to increased volume of effluents at the downstream end, thereby polluting the aquatic habitat [2]. Further, if there are any leaks due to increased hydraulic loads in the under-drainage network, then they may also contribute to recharge but degrade the quality of groundwater [3].

In order to create a balance between urbanization and environmental sustainability, there is a need for an integrated stormwater management system that minimizes the possibility of flooding, allows rapid and natural disposal of water through “infiltration”, and scales down the pollutant load [4]. The construction of Pervious Concrete Pavements (PCPs) in different parts of the world has emerged as a sustainable roadway material to mitigate the impact of urbanization and help develop eco-habitats [5, 6]. PCPs differ from conventional rigid pavements in the sense that the use of fine aggregates is minimized or totally eliminated. Pervious Concrete (PC) is typically a mixture of single-sized coarse aggregates and sufficient cement content with water to coat the aggregates and preserve mixture’s integrity [7]. However, several investigations have suggested that incorporation of finer particles in the mix in a selective manner can help achieve higher strength while maintaining the porosity requirements in the range of 15-35% [8, 9]. Due to the
absence of fine aggregates, PC material has low compressive strength of the order of 17-25 MPa and allows rapid infiltration of water through the pavement system. The typical infiltration rates in PC vary between 4.59 and 13.12 in/min, however, values as high as 27.56 in/min have been reported in the laboratory [10]. The infiltration rates in PC are significantly dependent upon porosity of the material, which in turn varies with cement-aggregate ratios, aggregate gradation, and water-cement ratios [11-14]. In a recent study, researchers showed that the porosity and permeability of PC decrease with increase in the cement-aggregate ratio due to a reduction in inter-granular void volume. Researchers suggest that increase in water-cement ratio leads to densification of the mix, which has a negative effect on the porosity of the material [11-13]. A mix composed of higher fractions of smaller sized aggregates provides more binder area and decrease the rate of permeability through the system [12, 14]. Further, the amount of compaction energy imparted to the PC material affects the strength characteristics and infiltration rates through the system. It is observed that higher compaction effort leads to reduction in the effective void content and infiltration capacity of the material [15].

Based on above studies, it can be concluded that attempts have been made in the past to study permeability properties of PC material in the laboratory. However, the investigation of drainability characteristics through PC material in the field is still a conjecture. The infiltration rate through PCPs in the field also depends upon the permeability of underlying base layers and nature of soil subgrade. Thus, the primary aim of this study was to estimate the infiltration of PCPs based on actual test sections constructed in the field. The scope of the effort of the study was twofold: measurement of field permeability through the surface and base course layers, and fitting a model to estimate the range of infiltration rates through the system.

2. MATERIALS

The interconnected porous matrix in the PC material is attained by either eliminating or minimizing the use of fine aggregates. The main purpose of this research was to estimate and assess the infiltration rate through the special material used for the construction of a parking lot so that it meets the standard of a sustainable construction. Ordinary Portland cement 53-grade was used in this study. Water-cement ratio was 0.32 and a polycarboxylic ether-based superplasticizer was added to the mix per the dosage prescribed by the supplier (0.6% by mass of cement) to improve the workability and enhance the bond between aggregate and cement. The aggregate gradations used by researchers in the past for preparing PC mix formed the basis for this study as well [8, 13]. Two distinct crushed aggregate combinations were procured from a local crusher plant. The aggregates from two lots were classified as 12.5 mm and down size, and 6.3 mm or lower sizes. The cement-aggregate ratio adopted was 1:3.75. Equal proportions of aggregates from the two lots were used to prepare a binary aggregate gradation. The particle size distribution curve for the two aggregate gradations used to construct pilot test sections is shown in Figure 1. As observed, the combined grading resulted in an open-graded proportion, which is the functional requirement of the PC material.
3. CONSTRUCTION OF PERVIOUS CONCRETE PAVEMENT TEST SECTIONS

3.1 Site selection
In March 2018, a PCP demonstration test section 4 m wide and 120 m long was constructed in the parking lot premises of Municipal Corporation of Tirupati, State of Andhra Pradesh, India. Figure 2 depicts the site condition before the start of construction.

3.2 Site preparation
Stringlines were extended and the soil was excavated up to a depth of 250 mm for preparation of the subgrade. The soil at the site was composed of sandy-gravel fill material and free of plastic fines that provided good structural support and sufficient permeability. Thus, the sandy-gravel soil fill was compacted with a 10 metric-ton static steel wheel roller to attain a flat surface. Twelve passes were accorded to achieve a compaction level of 95% of theoretical density. Figure 3 illustrates the steps involved in the compaction of subgrade.
Figure 3. Site preparation: (a) Marking boundary using stringline; (b) Process of the excavation of the ground; (c) Excavated soil at the site; (d) Compaction using static steel wheel roller

3.3 Pervious concrete pavement system

3.3.1 Preparation of sub-base

A typical PCP system consists of PC slab as the surface wearing course over a sub-base that is overlaid on a well-compacted subgrade. After the subgrade was compacted in the field, a sub-base layer of 200 mm thickness was laid upon it. The sub-base layer was composed of granular material conforming to the AASHTO 57 gradation available in IRC:58-2015 [16]. After placing the granular material, the sub-base layer was compacted with six passes using a static steel wheel roller. The basic purpose of providing a sub-base layer was to provide structural support to the PC slab and act as temporary storage medium before infiltrating stormwater into the subgrade. Figure 4 highlights the activities involved in the preparation of sub-base.

Figure 4. Sub-base preparation: (a) Laying of granular sub-base; (b) Compaction of the granular sub-base using static steel wheel roller
3.3.2 Preparation of formwork to measure infiltration rate

In order to compute the field infiltration rate through the PCP system, a wooden formwork with four sections (dimensions: 4 feet × 4 feet) was prepared and positioned towards the edge of the test section, such that it does not obstruct the vehicular movement across the mainline. The formwork was placed at the interface between subgrade and sub-base, and was given similar compaction efforts to simulate field infiltration rate in the mainline paving surface. Figure 5 shows the formwork placed in the field to compute field infiltration rates. Note that two circular pipes were placed below each wooden section across the length of the formwork and extended 150 mm outwards. Each pipe had 28 perforations of 6 mm diameter to drain the water during field infiltration tests.

![Formwork to measure field infiltration rate](image)

Figure 5. Formwork to measure field infiltration rate

3.3.3 Construction of pervious concrete slabs

Once the sub-base was compacted and leveled, PC slabs with a width of 4 m and thickness of 0.15 m were placed. The transverse joint spacing was 4 m. Full depth isolation joints were also provided and mastic pads (full width) were inserted between the PC slabs. The following lists the various stages involved in the construction of PC slabs (Figure 6):

- Setting up of line and grade
- Preparation of steel formwork
- Mixing of constituent materials in a drum mixer
- Transportation and placement of PC material
- Compaction using a 2 feet × 2 feet plate vibrator
- Curing for 28 days

By nature, PC dries at a very fast rate due to its open graded structure. Since the temperature in the vicinity was 41 °C during construction, water was sprayed over the freshly placed PC mix prior to compaction. Additionally, the PC mix was placed approximately 1-inch higher than the formwork and vibrated for about 90 s to achieve a flat surface in level with the formwork along with maintaining a uniform thickness of 0.15 m. The plate vibratory roller was slowly moved over a flat and stiff sheet placed over the PC mix to avoid direct contact with the surface. The compacted PC slabs were covered with plastic sheet and wet gunny bags for curing purposes. Figure 7 presents the final PC product along with close-up view of the surface texture.
Figure 6. Pervious concrete construction chronology: (a) Preparation of formwork; (b) Mixing and transportation; (c) Placement; (d) Vibratory compaction; (e) Alternate panel construction; (f) Final section before curing; (g) Curing process using plastic sheet and gunny bags; (h) Close-up view of the PC surface

Figure 7. Final sections of pervious concrete: (a) In-service test section; (b) Surface texture
4. FIELD INFILTRATION TESTS ON PERVIOUS CONCRETE PAVEMENT SYSTEM

In order to estimate the infiltration characteristics of the PCP system, an experimental setup was developed that could systematically study the percolation of water into the pervious concrete. The methodology adopted in this experiment was similar to a falling head permeability test for soil samples [17]. In order to facilitate ponding of water above the pavement, a wooden formwork was developed around the test section (see Figure 8). A falling head permeability test was performed on this test section to calculate the value of permeability of the entire section. In this test, water was ponded above the PC surface to a certain height, and the time taken for water to drain through the pavement was recorded. Graduations were marked on the wooden section from 0.15 m to 0 in intervals of 0.01 m.

![Figure 8. Cross-section details of PCP test section](image)

The cross-sectional area of the pavement, \( A \), through which the water drained was 1.49 m\(^2\). If \( Q \) is the discharge through the pavement resulting due to a reduction in water level ‘dh’ over a time interval ‘dt’, then:

\[
Q = -A \frac{dh}{dt} \quad \text{...(1)}
\]

Also, the discharge can be estimated using Darcy’s law as:

\[
Q = KiA \quad \text{...(2)}
\]

where, \( K \) is the permeability of the porous medium [LT\(^{-1}\)], \( i \) is the hydraulic gradient that drives the flow [LL\(^{-1}\)], and \( A \) is the cross-sectional area through which flow occurs [L\(^2\)]. By rearranging the terms in Equations (1) and (2), and integrating over time, the following equation was obtained:

\[
K = \frac{L}{t} \ln \left( \frac{h_1}{h_2} \right) \quad \text{...(3)}
\]

where: \( L \) is the thickness of the pavement system (see Figure 8), \( t \) is the time taken for water level to fall from \( h_1 \) to \( h_2 \).
4.1 Results and discussion:
In order to estimate permeability of the pavement sections, three trials of the falling head permeability test were performed as explained earlier. The results obtained are shown in Table 1.

<table>
<thead>
<tr>
<th>Height of water above PC surface (cm)</th>
<th>K (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Trial 1</td>
</tr>
<tr>
<td>h1</td>
<td>0.38</td>
</tr>
<tr>
<td>h2</td>
<td>0.27</td>
</tr>
<tr>
<td>55</td>
<td>0.38</td>
</tr>
<tr>
<td>54</td>
<td>0.27</td>
</tr>
<tr>
<td>53</td>
<td>0.38</td>
</tr>
<tr>
<td>52</td>
<td>0.32</td>
</tr>
<tr>
<td>51</td>
<td>0.24</td>
</tr>
<tr>
<td>50</td>
<td>0.22</td>
</tr>
<tr>
<td>49</td>
<td>0.25</td>
</tr>
<tr>
<td>48</td>
<td>0.25</td>
</tr>
<tr>
<td>47</td>
<td>0.25</td>
</tr>
<tr>
<td>46</td>
<td>0.23</td>
</tr>
<tr>
<td>45</td>
<td>0.23</td>
</tr>
<tr>
<td>44</td>
<td>0.22</td>
</tr>
<tr>
<td>43</td>
<td>0.19</td>
</tr>
<tr>
<td>42</td>
<td>0.16</td>
</tr>
<tr>
<td>41</td>
<td>0.14</td>
</tr>
</tbody>
</table>

It was observed that the permeability values of the pavement system showed a decline with time. A well-known analytical expression: Horton’s infiltration equation, which follows an exponential decay model, was used to find the best fit for the observed data. Horton’s equation expresses infiltration into a soil sample as follows:

\[ f(t) = f_c + (f_0 - f_c)e^{-kt} \] ……………………………(4)

\( f(t) \) = Infiltration Rate [LT\(^{-1}\)] as a function of time \( t \)
\( f_c \) = Equilibrium Infiltration rate \([\text{LT}^{-1}]\), after soil has been completely saturated

\( f_0 \) = Initial Infiltration rate \([\text{LT}^{-1}]\)

\( k \) = Decay constant \([\text{T}^{-1}]\)

Least squared errors optimization was performed between observed data and Horton’s model. The objective function was minimized for the following model parameters: \( f_c = 0.05 \text{ cm/s} \), \( f_0 = 0.29 \text{ cm/s} \), and \( k = 0.01 \text{ s}^{-1} \). Table 2 summarizes the infiltration rates of PC slabs with respect to their time intervals as observed in the field.

\[ \text{Table 2. Infiltration rates of PC with respect to their respective time intervals} \]

<table>
<thead>
<tr>
<th>Time taken from ( h_1 ) to ( h_2 ) (s)</th>
<th>Cumulative time, ( t ) (s)</th>
<th>Observed infiltration rate (cm/s)</th>
<th>Modeled infiltration rate (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.38</td>
<td>2.38</td>
<td>0.32</td>
<td>0.28</td>
</tr>
<tr>
<td>2.91</td>
<td>5.29</td>
<td>0.26</td>
<td>0.27</td>
</tr>
<tr>
<td>3.11</td>
<td>8.40</td>
<td>0.26</td>
<td>0.26</td>
</tr>
<tr>
<td>3.17</td>
<td>11.57</td>
<td>0.25</td>
<td>0.25</td>
</tr>
<tr>
<td>3.53</td>
<td>15.10</td>
<td>0.22</td>
<td>0.24</td>
</tr>
<tr>
<td>3.91</td>
<td>19.01</td>
<td>0.21</td>
<td>0.23</td>
</tr>
<tr>
<td>4.21</td>
<td>23.21</td>
<td>0.20</td>
<td>0.22</td>
</tr>
<tr>
<td>3.60</td>
<td>26.81</td>
<td>0.24</td>
<td>0.21</td>
</tr>
<tr>
<td>3.93</td>
<td>30.74</td>
<td>0.22</td>
<td>0.21</td>
</tr>
<tr>
<td>4.79</td>
<td>35.53</td>
<td>0.19</td>
<td>0.20</td>
</tr>
<tr>
<td>4.37</td>
<td>39.91</td>
<td>0.21</td>
<td>0.19</td>
</tr>
<tr>
<td>4.81</td>
<td>44.71</td>
<td>0.19</td>
<td>0.18</td>
</tr>
<tr>
<td>4.79</td>
<td>49.51</td>
<td>0.20</td>
<td>0.17</td>
</tr>
<tr>
<td>6.43</td>
<td>55.94</td>
<td>0.15</td>
<td>0.16</td>
</tr>
<tr>
<td>9.46</td>
<td>65.40</td>
<td>0.11</td>
<td>0.15</td>
</tr>
</tbody>
</table>

Figure 9 shows the observed and modeled infiltration rates as function of time. The values of infiltration rates in the PCP test section ranged from 0.05 to 0.29 cm/s.
The implications of the results obtained may be significant in the context of urban development, flash flooding, and groundwater recharge. For instance, the infiltration rate through soils range from $7 \times 10^{-6}$ to $3.5 \times 10^{-5}$ cm/s for clayey soils to as high as $7 \times 10^{-4}$ to $3.5 \times 10^{-3}$ cm/s for sandy soils. Compared to these, the infiltration rates for PCPs are higher by 2 to 4 orders of magnitude. Under natural conditions, soil may get saturated quickly, thus limiting recharge and increasing surface runoff. However, in case of PC pavements (when placed as parking lots), the high infiltration rates can allow for enhancing groundwater recharge significantly. Also, PCPs can drastically reduce flash flooding and soil erosion, which is usually the problem with traditional pavements. Thus, PC offers viable and sustainable solutions to address challenges posed by rapid urbanization.

5. CONCLUSIONS & RECOMMENDATIONS

The chief objective of the study was to estimate the infiltration capacity of the PCP pilot field test sections. Based on the study, following conclusions and recommendations can be drawn:

- Plate vibration for about 90 s at one stretch of the slab was found to be optimum. Excessive vibration was not provided since it may have created wet spots over the surface and lower vibrations may not be sufficient to achieve the necessary density. Also, the use of a hard sheet below the vibrator and above the PC surface was essential to avoid stripping of aggregates, and thus, a smooth riding surface was accomplished at the end of the task.

- It was found that during the falling head permeability test, the infiltration rate decreased with increasing time. The infiltration rate for PCP system in the field was observed to vary between 0.05 and 0.29 cm/s, which was higher by 2 to 4 orders of magnitude when compared to the infiltration rate through soils. Such higher infiltration rates of PCP may help reduce the occurrence of flash floods, enhance groundwater recharge, and help sustain a balance between rapid urbanization and environmental sustainability.
• Recommendations: while preparing PC mixes in the field, due care should be given to the mix proportions as the presence of excessive water content may cause the cement paste to run off from the aggregates; or low water content may render the mix too harsh. Since PC has exposed popcorn like structure with large number of voids, water should be sprinkled at regular intervals during placement prior to compaction in order to control the loss of moisture content. A curing compound is recommended to be used for PCP construction in future studies to control the rapidity of hydration through its porous network structure.

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REFERENCES

Analysis of For-Hire Passenger Vehicle Crashes in Namibia

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Extended Abstract

Namibia, like other low- and middle-income countries, is facing a public health crisis related deaths and injuries attributable to road crashes. The crash fatality rate in Namibia was recently estimated to be 23.9 per 100,000 population. Such a high rate of road deaths exacts serious social and economic stress on many countries, hence the need to critically examine and understand factors driving this trend, particularly among for-hire services vehicles. This category of vehicles constitutes a special group due to the high number of passengers they carry and the consequent potential for serious injuries resulting from crashes involving them. We analyze some 30,000 records of crashes that occurred in Namibia between 2010 and 2012, obtained from the National Road Safety Council. Preliminary data exploration revealed that more than 60% of for-hire vehicle crashes were single-vehicle crashes and a significantly high proportion occurred in built-up areas. About 8% of the crashes resulted in serious injuries. Ordered Probit model was developed to investigate the factors that influence the severity of crashes involving for-hire vehicles. Additionally, Latent Class Ordered Probit model was developed to reveal more details on how these factors vary across sub-populations of the crash data. Model estimation results show that for-hire vehicle crashes that occurred at night or on unlit roads and those that occurred in built-up areas were more likely to result in serious injuries. Also, the crashes involving vehicles that had more
than one passenger recorded some serious injuries. Crashes due to collision with wildlife were less likely to be severe. Crashes involving pedestrians were likely result in serious injuries in about 43% of crashes, while head-on collisions resulted in serious injuries in 57% of crashes. Female drivers were more likely to record no injury when they were involved in for-hire vehicle crashes. This study revealed the increased risk for passengers in for-hire vehicles and for pedestrians, particularly in built-up areas in Namibia. Strict enforcement of traffic rules is recommended to check reckless driving and jay-walking in built-up areas.
PAPER TITLE: Prevention of Overtaking Accidents on Rural Roads with the New German Guidelines for Rural Roads

TRACK

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KEYWORDS: Overtaking, Accidents, Rural Roads, Guidelines for Rural Roads, Risk analyses

ABSTRACT:

Overtaking accidents are very serious on German rural roads. The aim of the new German guidelines for rural roads is to avoid overtaking accidents on high volume roads with overtaking lanes. However, there are still many two-lane roads left. In two research projects the infrastructural and traffic related variables, which influence the number and consequences of overtaking accidents were analyzed. In order to derive appropriate measures to avoid these accidents diverse correlations between operational and infrastructural road characteristics and overtaking accidents were investigated and the main deficiencies were deduced.

In a comprehensive network analysis in five German federal states the most accident-prone two-lane rural road sections with relations to overtaking process were determined. As a result overtaking accidents occurs in a variety of road elements, which have a negative influence on overtaking sight distances (combinations of horizontal and vertical curves and also sight obstacles beside the road), but often there are no measures of traffic regulations. The number of overtaking maneuvers and overtaking density increases with better overtaking sight distances, but accident risk decreases significantly. In general, 600 m overtaking sight distance is needed. But 37 % of the overtaking accident were allocated in road sections with sight distances less than 300 m, 34 % between 300 m and 600 m sight distances and just 29 % in sections with sight distances above 600 m and thereby enough for safe overtaking. Generally, there are significant improvements of traffic safety when restrictions on overtaking or speed limits are placed.

Overtaking accidents are very serious on German rural roads. The aim of the new German guidelines for rural roads is to avoid overtaking accidents on high volume roads with overtaking lanes. However, there are still many two-lane roads left. In two research projects the infrastructural and traffic related variables, which influence the number and consequences of overtaking accidents were analyzed. In order to derive appropriate measures to avoid these accidents diverse correlations between operational and infrastructural road characteristics and overtaking accidents were investigated and the main deficiencies were deduced. Overtaking accidents occurs in a variety of road elements, which have a negative influence on overtaking sight distances (combinations of horizontal and vertical curves and also sight obstacles beside the road), but often there are no measures of traffic regulations. The number of overtaking maneuvers and overtaking density increases with better overtaking sight distances, but accident risk decreases significantly.
Prevention of overtaking accidents on rural roads with the new German guidelines for rural roads

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1 INTRODUCTION

In 2016, 75,266 accidents with personal injury on rural roads were registered by the police in Germany. Here, 1,853 persons died and other 25,841 persons were seriously injured as reported by DESTATIS (2018). About 6 percent of these accidents occurred due to overtaking maneuvers, but they cause approximately 8 percent of killed persons as analysed with data of DESTATIS (2018). This clarifies that overtaking accidents are one of the most serious accidents on German rural roads.

Due to such statistic values further research activities and analysis on accidents and their influencing factors are an essential part in the section of road planning and road design. The subsequent advancement of guidelines for the construction and operation of rural roads is a significant contribution for improving road safety. In the literature there are mainly older reports on overtaking accidents and overtaking behavior. For the changing of German guidelines for the design of rural road in 2012 new research had to be conducted to assemble actual values on the topic of overtaking on two-lane rural roads. Which road configuration leads to which overtaking behavior and which is critical for accident occurrence is the result of the mentioned project and theme of this paper.

The main result of the research activities was the finding, that there is a lack of unity of road construction (existing sight) and road operation (configuration of the traffic regulation). Within the complex process of overtaking the driver needs support from the road design in the task of driving to avoid errors and accidents. Moreover, a microscopic accident analysis was carried out to identify the essential facts of accident occurrence. Thereby five typical situations were detected and are involved in the further recommendations. In general, a set of influencing parameters for overtaking accidents and overtaking behavior are identified. Finally, this parameters of real overtaking behavior are used to evaluate possible measures empirically.

This paper contains an overall summary of the results of the above mentioned research project. Here, the findings of the literature, the macroscopic accident analysis, the influence of the configuration of the traffic regulation, the visibility analysis and evaluation of road layout, the detailed analysis of overtaking behavior and recommended measures to avoid overtaking accidents on two-lane rural roads are indicated, which are valuable informations for improving road safety. A scheme of the overall project approach and the content of this paper is depicted in Fig. 1.
1. Literature review

Overtaking is a very complex driving process with a variety of influencing factors. But the driver is physically and mentally not able to capture all the influencing factors rationally and make a decision based on a weighting. Even overtaking maneuvers under the same boundary conditions and with the same drivers will not be identically, see Netzer (1966). The traffic requirement for overtaking increases fundamentally with increasing speed dispersion in the traffic stream and generally with increasing traffic load. Missing overtaking possibilities can lead to accrued overtaking pressure which can cause risky overtaking maneuvers as reported by Steierwald et.al. (1983). Accumulation of overtaking maneuvers are correlating with existing overtaking sights and are influenced by the current traffic situation as mentioned in Kayser et.al. (1986).

Due to the complexity of overtaking and the overlay of unfavorable factors of drivers, vehicles and the driving environment there are numerous ways in which an overtaking maneuver can lead to errors and accidents. Structural and operational measures on rural road can improve the perception of difficulties and draw the drivers’ attention on problem areas, see Kämpf et.al. (2005). Kayser and Struif (1993) differentiated infrastructural measures against overtaking in positive-acting (additional overtaking lanes) and negative-acting measures (restrictions on overtaking and speed limitations). Within AOSI-project the positive effects on overtaking accident causation by providing sporadic safe overtaking opportunities (additional overtaking lanes) with intervening sections with restriction on overtaking already been proven and confirmed by Lippold et.al. (2012).

One core report for the new German guideline was Vieten et.al. (2010). The report underlines the safety effects of three and four lane rural roads with no overtaking in counterflow.

Table 1. Basic accident cost rate for different numbers of lanes and carriage width (Source: Bast Heft V 201, 2010)

<table>
<thead>
<tr>
<th>numbers of lanes and carriage width</th>
<th>Basic accident cost rate [Euro per 1,000 Vehicle-km]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two lanes, 7,00 m width</td>
<td>30,30</td>
</tr>
<tr>
<td>Two lanes, 8,00 m width</td>
<td>17,40</td>
</tr>
<tr>
<td>Three lane carriageway, 11,00-12,50 m width</td>
<td>11,30</td>
</tr>
<tr>
<td>Dual carriageway, four lanes, 14,50-16,50 m width</td>
<td>7,70</td>
</tr>
</tbody>
</table>

2. German guidelines for design of rural roads

Since 2012 there are new guidelines for the design of rural roads ‘RAL’ in Germany, see FGSV (2012). The general increase of traffic safety and also measures for the securing of overtaking maneuvers are some of the main aspects in this guideline. Since the introduction of ‘RAL’ the design of rural roads is based on four defined design classes with recommended design features (cross sections, road alignment, type of intersection, traffic regulation). The determination of a design class for a new road depends on the road role within the whole road network. To focus on the context of this paper, this definition of four design classes is connected with fundamental principles on overtaking, which aims to ensure overtaking on overtaking lanes or avoid them in sections with critical overtaking sights.

![Figure 2. Cross sections for the new German design classes on rural roads](image)
The design class with the highest traffic importance (EKL 1, see Fig. 2) secures overtaking maneuvers continually on an alternating middle overtaking lane in a three-lane cross section, which made approximately 40 % secure overtaking possibilities for each driving direction. On EKL 2 roads there are just partly overtaking lanes, which enable secure overtaking on approximately 20 % of each driving direction. Regarding the overtaking lanes there is a difference in road design compared to EKL 1 roads (see Fig. 2, EKL2/a). In the remaining two-lane parts of EKL 2 roads, driving in the lane of the oncoming traffic and finally overtaking is permitted by a line marking like in Fig. 2 (EKL2/b2). If there are insufficient sight distances for theoretically secure overtaking maneuvers overtaking can be prohibited by a central road edge marking (see Fig. 2, EKL2/b1). EKL 3 roads are conventional two-lane rural roads and generally the main part of the German road network outside of build-up areas. Here overtaking can be permitted in road sections with sufficient overtaking sight distances. On EKL 4 roads no needs for overtaking are designated, because they have the slightest traffic importance. All in all, overtaking in the lane of the oncoming traffic is generally possible on the two-lane sections at EKL 2 and EKL 3 roads with adequate overtaking sights by FGSV (2012).

Regarding the implementation of the new design class concept in the existing road network there are additional guidelines in development. Nevertheless, it can be assumed, that the implementation process of the new design classes on the existing road network will require a longer timeframe as mentioned in Richter and Zierke (2010). Therefore the task of the presented paper was to identify critical road features and tackle overtaking accidents for existing roads and also the new EKL 2 and EKL 3. Which design features and boundary conditions have to be considered for the prevention of overtaking accidents, is described in the context of this paper.

3. Macroscopic accident analysis

In a comprehensive road network analysis in the German federal states Baden-Wuerttemberg, Brandenburg, North Rhine-Westphalia, Rhineland-Palatinate and Saxony-Anhalt the most accident-prone road sections with relations to the overtaking process were determined. Therefore the accident databases of the years 2007-2009 were blended with the road information banks of the five federal states. Thereby 58,269 kilometers of two-lane rural roads were linked with 85,345 accidents with personal damage or serious property damage only.

The subset of overtaking accidents was identified through the supervening causes of the accidents, which are stated in the accident databases of the responsible police department. All in all, 6,200 overtaking accidents were filtered in the considered study area. For identifying the most accident-prone road sections the accident parameters were calculated and the accident cost density determined as selection criterion with the uppermost research capability. The result of this initially analysis was a ranking of the 500 rural road sections with the highest accident cost density of overtaking accidents which were used as a basis for further investigation steps.

3.1. Influence of road markings and signage

On the selected 500 road sections the present configuration of the traffic regulation (restrictions on overtaking and speed limits) were collected by own road inspections on a total length of 2,235 kilometers of rural roads. The evaluations showed that approximately on 70 percent of the investigated road sections overtaking is enabled and the remaining 30 percent were placed with restrictions on overtaking. If restrictions on overtaking are present, it is with 56 percent a marking (sign 295 German Highway Code), 22 percent a road sign (sign 276 German Highway Code) and the further 22 percent are a combination of road marking and signage. It has to be taken into account that a road marking by sign 295 German Highway Code just indicate that the drivers are not able to drive in the lane of oncoming traffic, so finally it is just an indirect overtaking restriction for e.g. two cars or a car and a trucks because of lane width restrictions. Nevertheless, cars and trucks are the main part of vehicles on rural roads, so finally this configuration of traffic regulation is also seen as a restriction on overtaking within this paper. After the road inspections a total of 1,557 overtaking accidents are directly assigned to the individual road characteristics (restrictions). 76 percent of the overtaking accidents happened in road sections without any regulation, but the lack of restrictions was examined later in the sight distance analysis. 24 percent of the overtaking accidents were found in existing restrictions on overtaking. The analysis of the positions of the overtaking accidents together with the surveyed configuration of the traffic regulation showed a slightly higher accident conspicuousness of released sections as they are present in proportion to the road network. Subsequently accident parameters were calculated to clarify the accident conspicuousness of the different configuration of traffic regulation. The results are given in Table 1.
Generally, there is a significant increase of traffic safety (vicarious through the calculated four accident parameters) when restrictions on overtaking or speed limits (main encountered configurations are 70/80 km/h, 50/60/90 km/h are very rare configurations) are placed in comparison to a decrease of safety in released road sections. The safety effects reach their maximum when both restrictions on overtaking and speed limits are ordered.

3.2. Influence of road design

In step of the research project a reproduction of the road layout at 100 accident-prone road sites had been carried out. This reproduction was the basis to check, which design parameters at accident locations are available and if the existing overtaking sights correspond to the configuration of traffic regulation on the road sections and whether the unity of road construction (existing sight) and road operation (configuration of traffic regulation) is ensured. To reach this, geographical point sequences were collected using GPS during the road inspections. These point sequences are rebuilt with a software for road design and the horizontal and vertical alignment of the road was recreated with all its design elements. Through an additional overlay with the road width and the lateral road design (implementation of roadside environment, sight obstacles and planting) a model of the road section arose.

The results of the road layout reproduction were section and sight tapes which were overlaid with the accidents and the configurations of the traffic regulation in order to obtain in depth information of accident-prone road characteristics and possible countermeasures. All in all, 350 kilometer of accident-prone road section were reproduced and 333 overtaking accidents had been allocated. The first conspicuousness’s concerning overtaking accidents and the rural road layout and special features are shown in Fig. 3. Generally, there was the fact that overtaking accidents happen in a variety of road elements, which have a negative influence on the existing overtaking sights (combinations of curves, vertical curves and sight obstacles beside the road). Out of the considered 333 overtaking accidents 236 accidents occurred in horizontal curves. Here, the load of accidents generally increased with decreasing curve radii. Right-hand bends (based on the right-hand driving in Germany) are a little bit more accident-prone compared to left-hand bends, because the obstacle vehicle in the traffic flow is an additional obstacle for the sight too.

![Figure 3. Overtaking accidents at different road elements](image-url)
Further 114 overtaking accidents occurred in vertical curves (superimpositions between horizontal and vertical curves are possible). Within this both geometric elements of road layout overtaking is permitted in more than 70 percent of the investigated overtaking accidents. Just 26 - 29 percent had a restriction on overtaking (marking and/or signage restriction). Moreover 39 overtaking accidents occurred within the influencing zones of intersections, where overtaking is generally prohibited. But this was just the case in 54 percent of the concerned accidents.

Another single influencing variable, which possibly have an effect on overtaking behavior and overtaking accidents, is the road width as reported by Palm and Schmidt (1999) and also Hegewald and Weber (2008). Within this project the cross-section design respectively the road width was also surveyed during the road inspections and matched to the standardized cross-section types of the last guideline generations for design of rural roads (RAS-Q 1982/1996), cf. FGSV (1982) and FGSV (1996). The reason for using just the older guideline generation was, that all considered road sections were planned in the past with these old guidelines. Within the matching process 57 percent of the considered road sections referred to the standardized cross-section types of RAS-Q (1996) and 30 percent to the standardized cross-section types of RAS-Q (1982). Further 13 percent of the road sections didn’t have any conformity to the standardized cross-section types in the past guideline generations. The result of the accident parameter calculation for all identified cross-section types is depicted in Fig. 4.

It is recognizable that the accident rates and accident cost rates in Fig. 4 decrease with increasing road width respectively wider standardized cross-section types. Nevertheless, there are no clear tendencies for accident density and accident cost density. Relating to the accident rates and accident cost rates it can be assumed, the merely the higher traffic volume and the higher traffic importance of wide cross-section types leads to this result, because traffic volume is considered with its reciprocal in the accident rate calculation.

Together with the change of the road design guideline generation in Germany and the introduction of the four design classes (see also chapter 3) there was also a change of the standardized cross-section types. Here the cross-section type for EKL 3 (RQ 11) has nearly the same cross-section design like RQ 10.5 of RAS-Q (1996) (cf. FGSV, 1996), excepting a slight extension of the shoulder width of 0.25 m on both sides. The two lane section of EKL 2 is also wider dimensioned, because there is a doubled central marking which leads to a further extension of 0.50 m. Both cross-section types are the safest in Fig. 4.

The cross-section type RQ 9, e2 with a road width of 6.00 m like in EKL 4 of RAL – cf. FGSV (2012) – belongs to the most unsafe cross-sections in Fig. 4. Nevertheless, for EKL 4 of RAL there are no regular overtaking maneuvers envisaged because of the new road marking type and the reduced planning speed of 70 km/h. Moreover, there would be a more centralized driving as reported by Zierke (2010) based on a before-after-comparison of driving behavior. That’s why it can be assumed that the rata of overtaking would be very low on this cross-section type. More specific statements on overtaking accident occurrence on EKL 4 are not deducible in this project just based on the road width because the type of road marking was changed.

Generally, the comprehensive analysis of overtaking accidents whilst taking into account the road width showed some slight coherences, but it can be assumed that the road width is not sufficient as the only influencing variable for describing the overtaking accident occurrence. If there are for instance two road sections with the same traffic conditions and equivalent road width but with differenced horizontal curvature, there will be a different overtaking behavior and accident occurrence (severity) too.

Finally, the results of road width showed, that the occurrence of overtaking accidents cannot be described with just one influencing variables. Consequently, the accident occurrence is dependent on a superimposing of horizontal and vertical road design as well as the cross-section elements, which results in the existing sight distances. This superimposing of different influencing variables is considered below.
accident because overtaking
Highway Code overtaking in the lane of the oncoming traffic should be prohibited, when the dangerousness of
600 meters, but the road operation conditions do not make the drivers aware of this. In accordance with the German
restrictions. This means that overtaking on cars and trucks is theoretically not possible with overtaking in areas with insufficient sights for normal overtaking maneuvers (below 600 m) but without any significant cause of such accidents. Regarding the configuration of traffic regulat
good sight conditions, what speaks for human misjudgments in the distance and speed of forthcoming vehicles as a
remaining 30 percent of overtaking accidents should theoretically be carried out safely due to
goal sight conditions, what speaks for human misjudgments in the distance and speed of forthcoming vehicles as a
result.

3.3. Influence of existing overtaking sight

After analyzing some single influencing variables, the main road layout elements had to be overlapped. The result of
the overlay of horizontal and vertical curvature, the road width and the lateral road design are the existing sight distances, which were calculated within the road layout model. While changing the German guidelines on rural road design, the definitions on necessary overtaking sights had changed. Before the year 2012 the necessary overtaking sights depends on the 85 percent quantile speed, cf. FGSV (1995) and FGSV (1980). Within FGSV (2012) there are just two boundary values for sufficient sights for long overtaking maneuvers (more than 600 meters for overtake a car or a truck) and short overtaking maneuvers (more than 300 meters for overtake a slow agricultural vehicle). Within existing sights under 300 meters (half overtaking sight) overtaking should be prohibited. In general, the standards for necessary overtaking sights within the two guideline generations are just different for speeds under 100 km/h. The project results in Fig. 5 depends on the old definition, the new boundary values are just a little bit stricter, but there is an obviously result.

It can be quantified, that 24 percent of the overtaking accidents occurred in areas with insufficient overtaking sights (less than half overtaking sight). Further 46 percent of overtaking accidents are in the range between the half and full overtaking sight. The remaining 30 percent of the overtaking accidents should theoretically be carried out safely due to good sight conditions, what speaks for human misjudgments in the distance and speed of forthcoming vehicles as a significant cause of such accidents. Regarding the configuration of traffic regulation, there are more than 73 percent of overtaking accidents in areas with insufficient sights for normal overtaking maneuvers (below 600 m) but without any restrictions. This means that overtaking on cars and trucks is theoretically not possible with overtaking sights less than 600 meters, but the road operation conditions do not make the drivers aware of this. In accordance with the German Highway Code overtaking in the lane of the oncoming traffic should be prohibited, when the dangerousness of overtaking cannot be discerned by the driver independently and therefore overtaking maneuvers cannot be performed because of safety reasons. Due to historical increased road network, there is a lack of road markings and signage, when accident black-spots were identified.

Figure 4. Accident Parameters of overtaking accidents differentiated by standardized cross-section types of RAS-Q (1982) and RAS-Q (1996): (a) accident rate; (b) accident cost rate; (c) accident density; (d) accident cost density.
Considering the element length of the configuration of traffic regulation and existing sights, the four accident parameters of Fig. 6 can be calculated. Generally, there are two main tendencies (just road sections with available annual average daily traffic data are considered). Firstly, the accident risk (in place of accident rate and density) and accident severity (in place of accident cost rate and cost density) decrease slightly with falling sights when no measures to avoid overtaking manoeuvres have been taken. Secondly through restrictions on overtaking obvious safety gains were appreciably in comparison to released road sections. In general, there are more sections with insufficient sights than sections with sufficient sights, that's why the accident parameters are decreasing, because there are a lot of sections without accidents, which were also considered in this network reflection. But there are two exceptions. The accident risk is nearly at the same level for sections without overtaking regulatory measures. Moreover and in consideration of the accident severity there are no differences between permitted and prohibited overtaking in sections with sufficient overtaking sights. The risk is lower, but if accidents occur they have similar consequences. Especially in the sections
4. Microscopic accident analysis

After completion of the sight tape analysis 50 road sections have been selected by means of a clustering, on which microscopic accident analyses and later detailed analyses of the overtaking behavior were carried out. The analysis of details of how the accidents occurred (police data and accident descriptions of 166 overtaking accidents) showed the following results.

In the most cases (97 percent) the overtaking vehicle caused the accidents solely, the remaining 3 percent were a partial blame. Therefore, the overtaking driver can be generally seen as the accident main responsible. This accident main responsible are mostly drivers of cars (82 percent), 13 percent motorcycles and 5 percent trucks. The decisive corresponding obstacles in traffic flow (vehicles which was overtaken) were by 54 percent cars, 31 percent trucks, 5 percent agricultural vehicles and 4 percent light motorcycles.

The further evaluation of the 166 accident descriptions results in 5 authoritative types of collision of overtaking accidents (visible in Fig. 7). Collisions with oncoming traffic are with 42 percent the largest group. Fundamental problems are here incorrect decisions in accepting or rejecting potential overtaking opportunities due to incorrect sight distance and speed estimation of the drivers. The other four influential collision types are more or less evenly distributed (the remaining two types are just exceptional cases, which are not considered here). These four collision types are collisions with turning vehicles (overtaking vehicle not recognized the turning intention), collisions with already overtaking vehicles (rear overtaking), collisions with road environment (loss of control while overtaking with sliding off the road) and collisions with the obstacle vehicle (swing out and also the process of going back into the own lane in connection with inadequate safety distances). Collisions with rear overtaking vehicles are with 52 percent with the involvement of motorcycles. Here, the high weight-acceleration-ratio compared with a car has the most negative influence together with insufficient orientation of the driver in the further overtaking vehicle. The collision opponents at overtaking accidents with left turning vehicles are with 55 percent cars, 22 percent agricultural vehicles and 15 percent trucks. Especially agricultural vehicles are problematic here because they often turning unexpectedly into inconspicuous agricultural roads.

![Figure 7. Collision types of overtaking vehicles.](image)

Some more facts can be mentioned as follows. With participation of motorcycles (comparatively vulnerable road users at rural roads) and trucks (high kinetic energy) the accident severity increases as expected. In general, 69 percent of the cases of accidents only one obstacle vehicle was overtaken. Further 19 percent of accident main responsible overtake two vehicles and at 12 percent more than two obstacle vehicles are stated directly in the accident descriptions. Overtaking a line of cars is seen as very dangerous, because the length of the necessary overtaking path is very difficult to estimate for drivers. Generally, 48 percent of the overtaking accidents occurred during the direct overtaking
manoeuvre (passing process), 27 percent during the swing out process and 19 percent during the process of going back into the own lane. For the remaining 6 percent none of the relevant phases of overtaking could be clearly assigned.

Young drivers are particularly prone to overtaking accidents. About half (46 percent) of accident main responsible are younger than 30 years. 85 percent of the overtaking accidents were caused by men. The proportion of overtaking accidents by contempt on existing restrictions on overtaking is with 53 percent the highest by novice drivers (under 20 years old). In general, for young drivers the main difficulties exist in the estimation of sufficient required length of overtaking paths, because of missing driving experiences. Older people tend to lacks of orientation in the traffic-related environment.

Finally, the identified basic problems of overtaking accidents are miscalculations of drivers (especially existing overtaking sights and also distance and speed of oncoming traffic), the loss of control, insufficient safety clearances and the lack of orientation in the surrounding traffic (conflicts with turning or already rear overtaking vehicles as well as conflicts at swing out and the process of going back into the own lane).

5. Analysis of overtaking behavior

During the detailed analysis of overtaking behavior, 10-hour surveys on rural roads were performed to get hints about the boundary conditions where overtaking maneuvers take place. Therefore a differentiation was made between road sections with and without overtaking opportunities (road sections with sufficient overtaking sights and by law permitted overtaking confronted with road sections with insufficient sights or by law prohibited overtaking). The main goal was to make relationships between the overtaking quantity and the road design characteristics. All in all 15,173 overtaking maneuvers in about 78 road sections were identified (some of the initial 50 road sections were subdivided by the above mentioned criteria) and associated with the road characteristics. The dividing of overtaking maneuvers into different car classifications (active/passive overtaking vehicle) resulted in decisive 34 percent car/car-overtaking maneuvers, 37 percent car/truck-overtaking maneuvers, 17 percent overtaking of cars at light motorcycles or agricultural vehicles and finally 7 percent motorcycle/car-overtaking maneuvers. The other combinations of vehicle classes are rather underrepresented. For the following evaluations just defined representative overtaking maneuvers are considered (subset of 12,315 overtaking processes). These are overtaking maneuvers which require a long overtaking path due to slight speed differences between the active and passive overtaking vehicle. These cases can be seen as the most critical maneuvers. As an example, car overtaking maneuvers at slow light motorcycles or agricultural vehicles are seen as comparatively uncritical due to high speed differences and consequential short overtaking paths.

As a further result, the number of accidents and representative overtaking maneuvers was reported for various design characteristics of the investigated road sections. Therefore the considered road sections were divided in the above mentioned three categories of existing overtaking sights (see Fig. 8a). Basically the number of overtaking maneuvers as well as the overtaking rate (considering the length of analysed road sections) increases with better visibility. Nevertheless, in all sections of existing sights overtaking accidents occurs. The connection of overtaking accidents and overtaking maneuvers quantify an accident risk per overtaking, so the reported accident risk decreases significantly with increasing sight distances (Fig. 8b).

![Figure 8. Overtaking accidents and maneuvers differentiated in: (a) overtaking sights; (b) relationship between both parameters.](image-url)
This potential risk is a value for estimating the risk of accidents per individually overtaking maneuvers. Thereby it has to be advised that the different periods which are under consideration (accidents within three years and overtaking maneuvers within ten hours) have to be considered. There would be the possibility to match both periods through extrapolation, but no matter which factor would be set, the tendencies will be the same.

In general, due to the weak overtaking behavior but nevertheless comparatively large numbers of overtaking accidents in sections with insufficient sights the accident risk during an overtaking maneuver increases. But even with only a few overtaking maneuvers a need of restrictions on overtaking arises to prohibit these maneuvers in sections with slight up to mean overtaking sights. Restrictions on overtaking revealed in total a significant decrease of overtaking accidents and maneuvers.

But unfortunately it has to be referenced, that road markings and signage as arrangements for traffic regulation can reduce the number of overtaking maneuvers but cannot invariably suppress the overtaking behavior. Independently from the kind of restrictions on overtaking 2,25 overtaking maneuvers per kilometer and hour could be ascertained in this sections during the empirical surveys. The amount of overtaking maneuvers in prohibited sections fluctuates dependent on other road boundary conditions like existing sight distance. The most drivers act on the given configuration of traffic regulation, but if there are just particular drivers who disregard the precepts there is a high danger of fatal overtaking accidents. Generally, acceptance problems for the chosen measures had to be declined here due to additional modular measures (e.g. additional speed limits by lasting accident black-spots).

6. Recommendations and measures tackling overtaking accidents

The recommendations of this paper are focused on infrastructural and operational measures to tackle black spots concerning overtaking accidents and general suggestions, where these measures should be applied. The results of this study clarified, that the existing overtaking sight is the road design feature with the main impact on overtaking safety. Moreover, the accident analysis points out a disunity of road construction (existing overtaking sights) and road operation (configuration of traffic regulation). Beside the complex process of situational assessment, the following weighting and the decision about acceptance or refusal of an overtaking possibility this lack of restrictions intensifies the overtaking behavior negatively. The network analysis yielded to a catalogue of specific measures, which are summarized below.

The risk of overtaking accidents is high. That’s why restrictions on overtaking are needed in road sections with insufficient overtaking sights (sights below the necessary full overtaking sight of 600 meters). With respect to slow driving vehicles (agricultural vehicles) the measures had to be divided in general restrictions on overtaking in sections with sights below the half overtaking sight (300 meters) and partially restrictions on overtaking with the release of overtaking at slow (e.g. agricultural) vehicles at intermediate sights (300 up to 600 meters). The restrictions had to be announced by arrow markings in the forefront to inform the driver early enough about the dangerousness of the following road section. These recommendations must be implemented strictly with renewed or generally newly constructed roads and at black spots on existing roads. But the results showed the need of additional measures at accident-prone road sections with already existing overtaking regulations too. Here, the efficacy of the existing measures has to be reinforced. Beside road markings and signage an additional speed limit can also reduce the risk of overtaking accidents and contain the overtaking behavior, because they reduce the speed dispersion and lead to a harmonized traffic flow. The safety impact of both measures was demonstrated within this report. Generally, all the above mentioned measures can improve traffic safety at black spots in the short-term.

Moreover, there are some other specifics of road design features, which have a negative impact on overtaking maneuvers too. Within the own road inspections there were road sections which attract attention, because they are equipped with existing restrictions on overtaking (road markings by dividing lines), but these restrictions are interrupted by broken lines for a length of 200 meters (with steady inadequate sights). Those interruptions had to be closed because drivers may think that overtaking is not prohibited anymore, so they have a potential possibility to overtake. This can lead to misunderstandings and causes accidents. Consistent and comprehensible measures are needed instead. At existing roads there is the possibility that available overtaking sights can be reduced by half abruptly, if there is an unfavorable functional interaction of horizontal, vertical and lateral road design. Those road sections must be restricted on overtaking maneuvers, because there is a high risk of overtaking accidents due to suddenly disappeared or appeared oncoming traffic behind sight obstacles.

Generally, there is a need of preventive measures within the range of influence of intersections to secure traffic flow at road sections where the overtaking pressure arises due to increasing speed dispersion (speed of the turning vehicle). The necessity of measures is heightened at inconsiderable road turn-offs.
Nevertheless, there are overtaking accidents in road sections with sufficient overtaking sights too. If there are accumulations of overtaking accidents in those sections there will be a need of so called positive-acting measures like additional overtaking lanes, where overtaking maneuvers can be performed safely. Those additional overtaking lanes are mostly long-term measures for rural roads with higher traffic importance and accordingly higher traffic volume. These measures require building works and cause corresponding costs.

Beside infrastructural and operational measures the driver education is important to highlight the general risk of overtaking maneuvers using the lane of the oncoming traffic, especially for young inexperienced drivers. If accumulations of disregards of existing restrictions are visible the enforcement plays an important part. Unlawful overtaking in ‘prohibited’ road sections must be punished hard, but there is still a lack of methods for its control.

7. Conclusion

Overtaking accidents are usually very serious accidents on rural roads. They occur mostly in road sections where overtaking is permitted. The analyses revealed that a large proportion of overtaking accidents occurs in areas with insufficient overtaking sights and where no configurations of traffic regulation have been taken to counter overtaking maneuvers. But the assumption that drivers can detect insufficient overtaking sights independently and therefore does not begin to overtake is wholly inadequate, because the complex weighting process of existing overtaking possibilities containing errors. Miscalculation of overtaking sights as well as speed and distance to oncoming vehicles are here the main problem areas. Missing configurations of traffic regulation can negatively warp the drivers’ perception. Instead, the drivers must be supported in road sections with insufficient overtaking sights through operational measures in their task of driving.

Several measures are discussed to prevent overtaking accidents. One measure is the consequently prohibition of overtaking with insufficient overtaking sight. The research projects have shown, that many car drivers didn’t respect the prohibition and an extensive enforcement is needed. Other discussed solutions are barriers in the center marking of the road like barriers. Challenges of these solutions are snow removal and blocking of the road by broken vehicles.

With the introduction of road design classes and the associated principles of overtaking in the guidelines for the design of rural roads in Germany (RAL) serious overtaking accidents can be avoided by clear precepts on overtaking and the safety of rural roads will increase. In design class 1 and 2, where frequently overtaking lanes are part of the concept, overtaking accident should be minimized. It is also recommended to use overtaking lines in design class 3 (regular two lane roads) not only for capacity reasons or at hills but also due to safety reasons to make overtaking safer at medium and high traffic load.

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KEYWORDS: Drivers’ behavior, classification of riskiness of drivers, traffic safety, Abu Dhabi, dangerous violations

ABSTRACT:
Drivers’ behavior has been extensively investigated in several prior studies. However, the number of studies that addressed the classification of riskiness of drivers (CRD) is relatively low compared to other drivers’ behavior types. In Abu Dhabi, there is no classification for the riskiness of drivers, but there are about 17 traffic violations considered as dangerous violations (DV). DVs represent about 18.6% of the total traffic violations in the last three years (from 2015-2017), and led to 41% of total serious accidents in 2017. In the last decade a significant improvement has been reached in the indicators of traffic safety in the Emirate of Abu Dhabi, even though the riskiness of drivers that contribute to the occurrence and severity of crashes in Abu Dhabi have not been explicitly classified in prior studies. This study aims to classify all drivers in the Emirate of Abu Dhabi for the purpose of following them and focusing on them. In addition, it aims to direct awareness programs appropriately, to direct reward programs and support good driver behavior in the right way, and encourage drivers to improve their traffic record through compliance with and application of road safety regulations.

The results indicated that there are four classifications for the degree of seriousness of traffic violations in Abu Dhabi (a traffic violation may result in some damage, a dangerous traffic violation may cause extensive damage, a very serious traffic violation can cause terrible damage, and a very serious traffic violation may cause catastrophic damage). Moreover, there are eight classifications of riskiness of drivers (ideal driver, perfect driver, very committed driver, committed driver, offensive driver, very offensive driver, dangerous driver, and very dangerous driver). Countermeasures to improve drivers’ behavior and reduce numbers and severity of crashes are also discussed.
CLASSIFICATION OF RISKINESS OF DRIVERS IN ABU DHABI

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1 INTRODUCTION

Driving is considered to be a complex task that needs drivers to apply a broader range of skills so as to interact with a challenging environment while managing other driving tasks simultaneously (Zheng et al., 2014). The task of driving or driver’s performance is not adequately modeled in any existing system; however, there are a number of driver riskiness and performance classification models/techniques linked to driver behavior in terms of risk perception and mental workload (Helfinstein et al., 2014). Thus, drivers might decide to select or follow various strategies for risky scenarios by considering different parameters such as speed of vehicles to perceived risk level. Additionally, there is no set of factors that can be applied in developing risk classification techniques encompassing behavior and performance of drivers. The classification technique of riskiness of drivers is simply a reflection of driver performance on roads that serve as a background for understanding characteristics that result in risky activities (Taylor et al., 2013). There are various driver performance or riskiness models designed to address a number of aspects regarding behavior of drivers on roads; however, the generally acceptable framework that encompasses all behavioral effects of drivers is unable to adequately describe the driving task (Wang et al., 2015). This is attributed to limited studies being conducted in this field. This section reviews some of the previous studies conducted in this field regarding classification of driver performance or riskiness in advanced countries.

A broader way of designing and developing a classification model for performance and riskiness of drivers is considered a combination of three primary factors – inputs, behavior of drivers and outputs. These elements might be used to explore or measure effectiveness of drivers or evaluate riskiness of drivers. Towfic (2014) affirms that the occurrence of road accidents might be reduced if there were reliable technology to detect any potential risky behavior exhibited by drivers. A technology that can identify characteristics of individual drivers who are responsible for poor or risky driving will significantly reduce driving errors and play a major role in improving road safety; this might be made possible through development of advanced vehicle assistance programs or tailored training for drivers. Wilde’s risk homeostasis theory explains how drivers try to maintain a certain risk level per unit of time; thus, if drivers are offered more safety measures, they are likely to show risky behavior so as to compensate to the targeted risk level (Mol, 2015). Different drivers have different levels of risk perception, which makes it critical to analyze and understand the performance of each driver. If such trends are categorized in various levels, a detailed model for risk and performance evaluation might be developed; however, a key drawback with such models is that they explain aspects influencing behavior of drivers without explaining the entire process of determining these characteristics of behaviors (Predic and Stojanovic, 2015). In addition, this technique does not have the ability to quantify elements associated with different levels of risk that might be used to develop statistical models.

Michon’s Hierarchical Control Model is among one of the most popular driver risky behavior models. This model proposed two different ways to classify and describe drivers’ behaviors (Medeiros-Ward et al., 2014). The first way that this model is suitable for distinguishing drivers’ behavior is through simple inputs and outputs. The second way is to differentiate functional models from taxonomic models where the components of the model might interact with each other. Michon’s model was subdivided to indicate three primary factors involved in decision-making of drivers; these factors include vehicle control, maneuvering and strategy (Škrjanc et al., 2017). Additionally, this model has been used as the basis of other studies in attempt to design a more effective riskiness classification model. A general assumption of Michon’s model is that the behavior or actions of drivers are fully dependent on the driving environment and their level of expertise. Drivers use perceptual and cognitive motor abilities to perform the driving task; thus, driving can be explained as a hierarchy of control,
navigation and guidance that is simultaneously conducted with recognition, monitoring operations and a visual search (Paaver et al., 2013). Based on this, driver’s personality, demands of tasks, state of mind and awareness of situations can be used to classify riskiness of drivers.

Abegaz et al. (2014) observe that the speed of a vehicle is critical when it comes to classification of driver’s riskiness; vehicle speed is important for traffic and road safety since it not only causes collisions, but also raises the risk of drivers ending up in collisions. Meiring and Myburgh (2015) conducted an in-depth review of studies that linked the speed of vehicles to the risk of drivers being in a collision. This research found that the risk of drivers getting involved in road accidents is likely to be higher for vehicles on urban roads than on rural roads. Moreover, Abegaz et al. (2014) found that higher vehicle speed in urban areas results in higher risk of collision or crashing. The risk of drivers being involved in road accidents is related to styles of driving. Driving styles are classified according to driver’s behavior and probability of risks on roads (Guo and Fang, 2013).

Lin et al. (2014) classify the riskiness and behavior of drivers based on five categories using the range of longitudinal closures. The data employed in this technique was classified by ANNs; once this network was developed, an index of aggressiveness was designed to show the frequency of a given driver overtaking and successfully passing other drivers. The system of numbering alongside the drivers’ age indicates the patterns and trends of behavior of drivers observed for various age groups. When this technique was used to show drivers’ behavior linked to tracking of the next vehicle, this technique never yielded desirable results; a major reason for not obtaining suitable results might be the limited input parameters of the network (Begum, 2013). Thus, since drivers’ behavior is significantly influenced by environmental and road aspects, the ANN network might be applied using different sets of parameters so as to predict and classify the behavior of drivers (Lin et al., 2014). There are other techniques or models that use the idea of vehicle dynamics, control theory and fuzzy logic. These models require prior assumptions and a significant amount of parameters to validate mathematical expressions (Karaduman et al., 2013). Furthermore, the models show reliable results but are highly dependent on assumptions created during their initial design and development. Thus, one key advantage of using ANN networks to model and determine driver’s riskiness and behavior is that there is no initial assumption made and its results might be obtained using a relatively smaller sample of input parameters. Neural networks have been used successfully in this field of driver riskiness and behavior because they can capture different driving aspects through a training process using adjustable parameters to achieve the required results (Van Ly et al., 2013).

The main objective of this paper is to classify all drivers in Abu Dhabi for the purpose of following them and focusing on them. In more detail, the study aims to direct the awareness programs appropriately, to direct reward programs and support good driver behavior in the right way, and encourage drivers to improve their traffic record through compliance and application of road safety regulations.

2 METHODOLOGY

A focus group was used to realize the objective of the study. As noted by Hart (2008), a focus group is a type of qualitative research method where the researcher identifies a group of people within a research setting from whom data can be collected based on a peculiar characteristic common to all these people. The selection of focus group meant that the opinions, views, and judgment of the respondents was going to be taken as the outcome of the group and acted upon on behalf of the entire group instead of acting on individual outcomes from the respondents. A panel of experts, namely, twelve traffic police officers with more than five years’ experience and professional line of work at the traffic safety in Abu Dhabi were identified to be a sample of this focus group. The issue that was to be addressed from the perspective of the experts was to develop a classification of riskiness of drivers in Abu Dhabi, and can be used to direct the awareness programs appropriately, to direct reward programs and support good driver behavior in the right way, and encourage drivers to improve their traffic record through compliance and application of road safety regulations.

The focus group was carried out in the form of an expert panel, which was qualitative in nature, and the panel was engaged in a collective manner rather than individually. Given the fact that the focus group was used for the purpose of creating the classification of riskiness of drivers in Abu Dhabi, it was needed to collect data that was based on human opinion, not just concepts with numbers. As the persons engaged in the focus group were experts who had more than five years of experience in the area of traffic safety, the outcome was considered as reliable. On the other side, this method had its own limitations, because the researcher had to serve their interests together and engage the respondents as a group, which makes it not easy to use a very large size of respondents (12 respondents).
As part of the effects of ensuring the validation of the classification of riskiness of all drivers in Abu Dhabi, a questionnaire-based survey was used. A panel of experts, namely, ten policemen with more than 15 years’ experience and professional line of work in the Traffic and Patrols Directorate were identified to be a sample for this questionnaire. The issue that was to be addressed from the perspective of the experts was whether the use of the designed classification could help to follow risky drivers and focus on them. The questions used focused on the investigation of the relationship between the usage of designed classification and directing the awareness programs appropriately, to direct reward programs and support good driver behavior in the right way, and encourage drivers to improve their traffic record through compliance with and application of road safety regulations.

3 DATA ANALYSIS AND DISCUSSION

Figure 3.1 below illustrates the statistics of dangerous traffic violations and non-dangerous violations between 2015 and 2017. In 2015, there were 1,316,657 cases of dangerous violations and 4,476,196 cases of non-dangerous violations, resulting in 5,792,853 total cases. Cases of dangerous traffic violations represented about 28% of the total violations in 2016. In 2017, there were 181,565 dangerous traffic violations which is the lowest in the last three years and represent about 4% from 4,637,251 which is the total violations in that year. The main reason for the dramatic decrease in dangerous violations is the new classification for the dangerous violations while there are many types of violations which were considered normal violations in 2017 up to the revision of the violations that caused the accidents.

Figure 3.1: Dangerous and non-dangerous traffic violations

From Table 3.1 below, it can be seen that there were various causes of series accidents between 2015 and 2017. Sudden swerve, failure to leave enough distance, not giving way for other road users, failure to comply with the lanes, and speed without taking into account the road conditions were the major five causes of serious accidents between 2015 and 2017 in Abu Dhabi. Generally, total traffic violations decreased significantly between 2015 and 2017 – this can be linked to improved road conditions and traffic regulations.

<table>
<thead>
<tr>
<th>Table 3.1: Causes of serious accidents in Abu Dhabi</th>
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<tr>
<td>Cause of accidents</td>
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<tr>
<td>Sudden swerve</td>
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<td>Failure to leave enough distance</td>
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<td>Not give way to users of the road</td>
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<td>Failure to comply with the road line</td>
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<td>Traffic Violation</td>
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<td>Speed without taking into account road conditions</td>
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<td>Neglect and lack of attention</td>
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<td>Jumping a red light</td>
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<td>Not giving way to pedestrians on pedestrian crossings</td>
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<td>Driving under the influence of alcohol, drugs or similar substances</td>
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<td>Entering main road without ensuring that it is clear</td>
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<td>Lack of driving</td>
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<td>Not stopping at a stop sign</td>
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<td>Driving without a driver's license</td>
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<td>Not giving way / not giving priority</td>
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<td>Explosion of tire</td>
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<td>Reversing without making sure the road is free</td>
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<td>Fatigue and sleepiness</td>
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<td>Driving the wrong way (opposite direction)</td>
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<td>Exceeding speed limit</td>
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<td>Reverse without attention</td>
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<td>Not aware of the road</td>
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<tr>
<td>Vehicle malfunction</td>
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<tr>
<td>Entering road dangerously for trucks or other vehicles</td>
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<tr>
<td>Road malfunction</td>
</tr>
<tr>
<td>Reckless driving</td>
</tr>
<tr>
<td>Impulsive vision</td>
</tr>
<tr>
<td>Sudden stop</td>
</tr>
<tr>
<td>Non-compliance with traffic signals</td>
</tr>
<tr>
<td>Not securing vehicle while parked</td>
</tr>
<tr>
<td>Vehicle malfunction</td>
</tr>
<tr>
<td>Prohibited entry</td>
</tr>
<tr>
<td>Using a mobile phone while driving</td>
</tr>
<tr>
<td>Failure to take the necessary traffic safety steps when vehicle breaks down</td>
</tr>
<tr>
<td>Not giving way for vehicles to pass on the left</td>
</tr>
<tr>
<td>Falling or leaking load</td>
</tr>
<tr>
<td><strong>Total</strong></td>
</tr>
</tbody>
</table>

From the above results, we see it is very important to have a new technique to deal with both committed drivers and dangerous drivers. Deal with committed drivers by encouraging them and rewarding them, and deal with dangerous drivers by punching them. The twelve traffic police officer experts were brought together with a moderator to focus on how we can classify the riskiness of all drivers in Abu Dhabi. All focus group members forecasted that we should have a new technique to classify the riskiness of drivers related to their traffic violations record.
The results indicated that there are four classifications for the degree of seriousness of traffic violations in Abu Dhabi:

1. Traffic violation may result in some damage (Level four),
2. Dangerous traffic violation may cause extensive damage (Level three),
3. Very serious traffic violation can cause terrible damage (Level two),
4. And a very serious traffic violation may cause catastrophic damage (Level one).

From this point, all violation clauses in Abu Dhabi were classified to level up to the severity as shown in Table 3.2.

<table>
<thead>
<tr>
<th>No.</th>
<th>Severity level</th>
<th>Description</th>
<th>No. of violation clause</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Level four</td>
<td>Traffic violation may result in some damage</td>
<td>16  19-A&amp;B  21  25-A&amp;B  55-A&amp;B  61</td>
</tr>
<tr>
<td></td>
<td></td>
<td>71  75  78  79  87  105-A</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Level three</td>
<td>Dangerous traffic violation may cause extensive damage</td>
<td>8  10  18  20  22-A  29</td>
</tr>
<tr>
<td></td>
<td></td>
<td>30  32-A&amp;B  36  41  43  48</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>51-A  52  53  54  56  73</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>100-A&amp;B  101  104-A  105-B  -  -</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Level two</td>
<td>Very serious traffic violation can cause terrible damage</td>
<td>9-A&amp;B  11  18  22-B  24  31</td>
</tr>
<tr>
<td></td>
<td></td>
<td>33-A&amp;B  35  42  69  76  90</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>91  104-B&amp;C  -  -  -  -</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Level one</td>
<td>Very serious traffic violation may cause catastrophic damage</td>
<td>1  2  3  4  5-A&amp;B  6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7  17  34  45  46  47</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>50  77  -  -  -  -</td>
<td></td>
</tr>
</tbody>
</table>

The ideas collected are used in making refinements to the product of this study. With the help of focus groups, the researcher collected information pertaining to what different groups or a set of people feel about how the riskiness of drivers can be classified. Feedback is collected from a focus group in case the classification changes the quality of the dealing with different drivers in the roads.

Furthermore, as a result of this focus group, the researcher established eight classifications of commitment/riskiness of drivers (ideal driver, perfect driver, very committed driver, committed driver, offensive driver, very offensive driver, dangerous driver, and very dangerous driver) depending on their traffic violation history. Also, to make them easier to classify, all eight categories were put into four categories and each was given a rank of stars (five stars is platinum driver, four stars is gold driver, three stars silver driver, and the fourth category is a dangerous driver). Based on the responses, the researcher set out the results as in Table 3.3.
Table 3.3 Classify the degree of commitment of drivers

<table>
<thead>
<tr>
<th>No.</th>
<th>Driver classification/rating</th>
<th>Level description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Platinum ★★★</td>
<td>He/she has not committed any traffic violations in the last three years and has not committed any violations first and second level in his/her life</td>
</tr>
<tr>
<td></td>
<td>★★</td>
<td>Ideal driver</td>
</tr>
<tr>
<td></td>
<td>★</td>
<td>Perfect driver</td>
</tr>
<tr>
<td>2</td>
<td>Gold ★★★★</td>
<td>He/she has not committed any traffic violations for the last three years and has not committed any violations in his/her life of the first level</td>
</tr>
<tr>
<td></td>
<td>★</td>
<td>Very committed driver</td>
</tr>
<tr>
<td></td>
<td>★+</td>
<td>Committed driver</td>
</tr>
<tr>
<td>3</td>
<td>Silver ★★★</td>
<td>He/she committed a level 4 violation in the last year at a rate not exceeding 10</td>
</tr>
<tr>
<td></td>
<td>★</td>
<td>Offensive driver</td>
</tr>
<tr>
<td></td>
<td>★</td>
<td>Very offensive driver</td>
</tr>
<tr>
<td>4</td>
<td>Dangerous</td>
<td>He/she committed a level 4 violations at a rate more than 20 or committed a level 3 violations at a rate more than 5 violations or committed any violation form level 2 in the last year or committed any violation in level 1 in the last 3 years</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td>Dangerous driver</td>
</tr>
</tbody>
</table>

The practical use of the above classification will be through the electronic driving license, which will be available for official use, insurance companies, and personal use. The color of the electronic driving license will be up to the degree of the classification (platinum driver = white color, gold driver = yellow color, silver driver = silver color, and dangerous driver = red color). Also, the stars and the rate (for example: Platinum driver ★★★★★ (A+ Ideal driver)) will show on the electronic driving license.

4 VALIDATE THE CLASSIFICATION OF RISKINESS OF ALL DRIVERS IN ABU DHABI

Because of the need to correct the situation, it was important that the researcher will validate the established classification of the riskiness of all drivers to be assured that it could help control the danger on Abu Dhabi roads. The validation process was well planned and thus based on the research questionnaire design. The respondents were made up of ten traffic safety policemen, all of whom have more than 15 years of working experience in traffic safety in Abu Dhabi (their experience is 216 years). On the whole, the questionnaire sought to find how appropriate the classification for all drivers in Abu Dhabi was to follow them, direct the awareness programs appropriately, direct reward programs and support good driver behavior in the right way, and encourage drivers to improve their traffic record through compliance and application of road safety regulations.

The questionnaire questions were divided into three groups. The first one was related to the current situation and it is clear in questions 1, 2 and 3 that there was a great endorsement in these questions that the current case is not satisfactory. The second group of questions (questions 4, 5 and 6) were related to the importance of establishing a new classification for the commitment of drivers in Abu Dhabi and the results show high support for this direction. The third category of questions (7 & 8) related to the designed classification and it has been a unanimous decision. The opinion of respondents on the validation of the designed classification have been presented in Table 4.1 below.
## Table 4.1 Validate the designed classification

<table>
<thead>
<tr>
<th>No.</th>
<th>Question</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Current techniques to classify the drivers’ commitment in Abu Dhabi satisfactory</td>
<td><img src="neither" alt="Disagree" /></td>
</tr>
<tr>
<td>2</td>
<td>Awareness and reward programs for drivers currently are appropriately targeted to improve traffic safety in Abu Dhabi</td>
<td><img src="neither" alt="Disagree" /></td>
</tr>
<tr>
<td>3</td>
<td>Currently, appropriate tools are available to support good driver behavior and to encourage drivers to improve their traffic record through compliance with traffic regulations and laws</td>
<td><img src="neither" alt="Disagree" /></td>
</tr>
<tr>
<td>4</td>
<td>The importance of establishing a classification for the commitment of drivers in Abu Dhabi</td>
<td><img src="neither" alt="Disagree" /></td>
</tr>
<tr>
<td>5</td>
<td>Do you believe that traffic safety management through activation of the classification of the riskiness of drivers in Abu Dhabi will contribute to increasing traffic safety in the Emirate?</td>
<td><img src="neither" alt="Disagree" /></td>
</tr>
<tr>
<td>6</td>
<td>Do you believe that traffic safety management through activation of the classification of the riskiness of drivers in Abu Dhabi will support drivers to improve their traffic record through compliance with traffic regulations and laws?</td>
<td><img src="neither" alt="Disagree" /></td>
</tr>
<tr>
<td>7</td>
<td>Do you believe that the activation of the classification of the riskiness of drivers in Abu Dhabi that designed this study can help to track and focus on risk drivers and appropriately guide awareness programs to increase traffic safety in the Emirate?</td>
<td><img src="neither" alt="Disagree" /></td>
</tr>
<tr>
<td>8</td>
<td>Do you believe that the activation of the classification of the riskiness of drivers in Abu Dhabi that designed this study can help to direct reward programs and support good driver behavior in the right track to increase traffic safety in the Emirate?</td>
<td><img src="neither" alt="Disagree" /></td>
</tr>
</tbody>
</table>
5 CONCLUSIONS

In conclusion, it could be said that the current techniques to classify the drivers’ commitment, awareness and reward programs, and available tools to support good driver behavior and to encourage drivers to improve their traffic record through compliance with traffic regulations and laws in Abu Dhabi is not satisfactory. There is a persistent need for a new technique to classify the riskiness of drivers related to their traffic violations record. In this study, the seriousness of all traffic violations in Abu Dhabi was classified in four levels. Also, eight classifications of commitment/riskiness of drivers (ideal driver, perfect driver, very committed driver, committed driver, offensive driver, very offensive driver, dangerous driver, and very dangerous driver) were established depending on their traffic violations history. Also, to make them more easy in classification, all eight categories were put into four categories and each of them was given a rank of stars.

The use of the designed classification will be through the electronic driving license, which will be available for official use, insurance companies, and personal use. The color of the electronic driving license will be up to the degree of the classification and the stars will be shown in the electronic driving license. Given the great endorsement and validation of the designed classification by using questionnaires, this new technique can direct the awareness programs appropriately, direct reward programs, and support good driver behavior in the right way, as well as encouraging drivers to improve their traffic record through compliance and application of road safety regulations. All these will have a positive impact on the overall safety performance of all Abu Dhabi roads.

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# All-Round Methodology for Upgrading Highways Prone to Accidents Using Virtual Driving

## Keywords:
- highway
- safety
- driving simulation
- driving behavior
- methodology

## Abstract:

The future focus of roadwork in Germany, where there is a high density of 1.6 km of road per km² of land, will involve upgrading and rebuilding rural highways prone to accidents. As new guidelines are not completely relevant for upgrading existing highways and exceptional permits are often required, there is an increasing need to use driving simulation in the design process to assess discrepancies from the standards and their effect on traffic safety. The highway is automatically retraced using survey data and genetic algorithms in stage 1 (preliminary review) and is broken down into the horizontal and vertical projection design elements. Stage 2 involves determining the shortcomings on the highway by superimposing the design elements, the actual accident figures and virtual driving runs. The actual replanning work occurs in stage 3, taking into account the minimum and maximum standard figures for the design elements.
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1 THE PROBLEM

Germany has a network density of 1.6 km of rural roads (highways and rural roads) per km² of its territory and is therefore one of the countries in Europe with a very well developed road network. New building works are restricted to closing gaps in the highway network (network length: approx. 12,800 km) and building ring roads and relocating roads in the rural road network (network length: approx. 218,000 km).

As approx. 66% of all driving accidents and accidents in parallel traffic occur on existing rural roads, the main task in road design in Germany during the next few years will involve upgrading existing roads that are prone to causing accidents.

As today’s rural roads have developed over centuries from former tracks on the boundaries of fields to paved routes for horse-drawn carriages and now to roads for powered vehicles with high driving speeds, any problems in the route characteristics (alignment on the horizontal and vertical projections and the cross section) are increasingly causing accident black spots, as drivers are often unable to adapt their driving behavior to the changes in the road situation at short notice.

Overtaking bans, speed limits and other passive measures are normally not very successful, because vehicle drivers often respond in an uncontrolled and inappropriate manner in heavy traffic (driving too fast, risky overtaking maneuvers etc.).

The same rules apply in Germany for upgrading roads as for new construction work. Because of the existing situation, it is often not possible to comply with the design parameters because of constraints (buildings, the environment etc.) or for economic reasons. This results in applications for exceptions, which are only accepted by the public authorities if professional justification is provided without any major safety risks. This process is often long-winded, unpredictable and largely depends on the responsible expert who is called in.

2 THE AIM

In order to be able to standardize the modification and upgrading work on rural roads and complete it in a more cost-effective manner, a comprehensive methodology is required, which requires an approach involving several stages. The necessary modification and upgrading sections within the existing road must be localized using a superimposition process of the shortfall areas in the alignment (horizontal and vertical projections and the three-dimensional alignment and cross section) with the accident black spots (driving accidents and accidents in parallel traffic) and with the problems in driving behavior (virtual driving). The design parameters of the current design guideline for rural roads (1) should be used to assess deficits in the alignment. To objectively assess the problems in driving behavior, quantitative parameters can be derived from the speed, road position and steering wheel movement parameters.

After localizing the modification and upgrading areas, they should be replanned as close to the existing road as possible, taking into account the current restrictions. If any deviations from the maximum and minimum figures for individual elements and sequences of elements are necessary according to the standard rules, the results of virtual driving operations should be used as a professional aid to making decisions for approving exceptions.

The methodology must be tested on selected examples, validated and prepared for introduction into design practice.
3 THE CURRENT STATE OF SCIENTIFIC KNOWLEDGE AND TECHNOLOGY

An analysis of the design guidelines available internationally (2) has shown that driving simulation does not yet form part of the process of assessing driving behavior and generally analyzing safety.

Here is a summary of the most important factors for developing the new kind of methodology for upgrading existing rural roads:

- The most important goal in the planning process for modifying and upgrading building work is traffic safety, which can be described in terms of the accident situation on a road. A harmonious speed curve with speed differences of \( \leq 10 \text{ km/h} \) should be aimed for between individual elements and road sections (3, 4).

- The driving behavior can be described largely through the vehicle’s position on the road, in addition to the speed curve. The driver should be able to stay in his or her lane during the journey (5).

- Several European countries make use of a so-called operating, development and planning speed for the checking process on the alignment (6, 7, 8, 9), i.e. a check of the expected speed curve process makes sense as early as the design phase.

- The figures for the minimum curve radii depending on the transverse gradient and the design speed are very different in individual countries. Germany tends to require larger radius figures.

- There is also clear evidence that the driving behavior and therefore the speed curve on a road largely depend on the sequence of elements, the ratio between radius/radius and radius/straight length. While the ratio between adjacent radii is 1:1.5 to 1:2 in Germany (1) and Italy (10), it is possible to have a ratio of 1:3 in Canada (11).

Based on an analysis of the literature, it is possible to make the following basic statements for drawing up the new kind of methodology:

(1) Driving behavior can be objectively described using the speed curve and road position. A check on these two factors therefore makes sense during the planning process by means of driving simulation.

(2) The radius ratios must be followed when designing bends, if the minimum radius is not met.

4. DESIGN METHODOLOGY

4.1 Summary

Based on the special requirements for modifying and upgrading existing roads, a fundamental methodology has been developed within the scope of a research project (12) (Figure 6). The whole process takes place within 3 stages:

- **Stage I: Preliminary checks**
  After drawing up digital measurement documents of the existing road, the design parameters are set depending on the design speed \( v_E \) (in Germany the design class [EKL]). The major focus of the design technology work within this stage 1 is on rerouting the existing road by determining the existing design parameters.

- **Stage II: Analysis of existing route**
  After localizing the accident black spots by assessing the accident type maps, checks are made on the existing design elements on the horizontal and vertical projections (individual elements, sequences of elements) in line with the German Design Guidelines for Rural Roads (RAL). Any deviations from the standard rules are noted. The existing road is “driven along” on the driving simulator for the purpose of establishing the driving behavior. Any problems in the driving behavior are localized and marked using feature graphs. By superimposing the accident black spots with the shortfalls in the alignment and the
problems in driving behavior, it is possible to establish the route sections where the alignment urgently needs to be changed as part of the modification and upgrading planning work.

- **Stage III: Modification and upgrading process**

The replanning process involves an iterative improvement along sections of the alignment. This gives rise to different options with different deviations from the design parameters in the standard rules. Virtual driving sessions are used on the driving simulator to assess the driving behavior and safety. If a route deviates from the standard rules in certain sections, but the driving behavior is harmonious, no further optimization may be necessary and this confirms the upgrading option. Quantifiable parameters are increasingly being used to objectively assess driving behavior.

![Diagram of the replanning process]

Figure 1. Basic methodology for upgrading existing rural roads

4.2 Rerouting work using the HighwayDesigner software

4.2.1 **Mathematical approach**

The classic design elements from the horizontal projection – straights, clothoids, circular arcs – are superimposed with those from the vertical projection – rising gradients and descents (straights) or crests and sags (quadratic parabola) – during the calculation process in real time in order to portray the route in a three-dimensional form.

The horizontal projection curve is made up of the classic design elements with the following individual approaches (Figure 2):
Figure 2. Horizontal projection curve (axis)

\[
\begin{align*}
\text{Straight:} & \quad f_G(t) = (1 - t) \cdot T_i + t \cdot T_{i+1} = Q_{LP}, \text{mit } t = g(s) \\
\text{Clothoid:} & \quad f_R(t) = \left( \frac{A}{\sqrt{2}} \cdot \int_0^t \frac{1}{2} \cdot \cos \tau \, d\tau - \frac{1}{2} \cdot \sin \tau \, d\tau \right) = Q_{LP}, \text{mit } \tau \cdot \frac{S}{2 \cdot R} \\
\text{Circular arc:} & \quad f_{RB}(s) = \begin{pmatrix} x_M + R \cdot \cos \left( \frac{s}{R} \right) \\ y_M + R \cdot \sin \left( \frac{s}{R} \right) \end{pmatrix} = Q_{LP}
\end{align*}
\]

where

\begin{align*}
s & \quad \text{location point} \\
g(s) & \quad \text{transformation function} \\
T_i, i = 0(1)n & \quad \text{tangent interpolation point} \\
x_M, y_M & \quad \text{center of the circle} \\
R & \quad \text{radius of the circle} \\
A & \quad \text{clothoid parameter} \\
\tau & \quad \text{tangent angle} \\
Q_{LP} & \quad \text{calculated point on the horizontal projection}
\end{align*}
The gradient (Figure 3) is the result of combining the following design elements: longitudinal incline (straight) and vertical curve (quadratic parabola).

![Diagram of gradient](image)

**Figure 3.** Vertical projection curve (gradient)

**Straight:**  
\[ f_S(s) = m \cdot s + n = Q_{HP} \]  
(4)

**Clothoid:**  
\[ f_P(s) = \frac{s^2}{2 \cdot H} = Q_{HP} \]  
(5)

where

- \( s \): location point
- \( m, n \): parameters for the straights
- \( H \): sag or crest vertical curves
- \( Q_{HP} \): calculated point on the vertical projection

### 4.2.2 Calculation model

The three tangent interpolation points \( T_i, T_{i+1}, T_{i+2} \) and the associated angle of direction \( \gamma \) are required to calculate the sequence of elements: straight \( (f_S) \) – combined curve \( (f_{HC}) \) – straight \( (f_S) \). The calculation of the local combined curve takes place with this angle and the vectors \( \vec{a} = T_i - T_{i+1} \) and \( \vec{b} = T_{i+2} - T_{i+1} \) (Figure 4).
Coordinate transformations are necessary in order to be able to transfer the local coordinates to the three-dimensional dimension. Then $T_{i-1}$ serves as the reference point for the shift and the angle $\beta$ is used to rotate the combined curve. The following transformation matrices are used for the overall transformation process:

$$Q_{12}(s) = M_{RST} \cdot M_{Trans_{i+1}} \cdot \begin{pmatrix} x_{Global} \\ s \\ z_{Global} \end{pmatrix}$$

(7)

The parameter $s$ is the length of the route.
The suitable function is selected from the list of vertical projection elements for each location point $s$ in

$$f(s) = \begin{cases} f_{GHP} & \text{if } f_{GHP} < f_{P(HP \text{ in})} \\ f_{P(HP \text{ in})} & \text{otherwise} \end{cases}$$

(8)
order to calculate the height. Then the superimposition takes place with the points $Q_{2D}(s)$ previously calculated on the horizontal projection (Figure 5):

$$Q_{3D}(f(s)) = \begin{pmatrix} X_{Global} \\ Y_{Global} \\ Z_{Global} \end{pmatrix} = \begin{pmatrix} x(s) \\ y(s) \\ z(s) \end{pmatrix}$$

(8)

4.2.3 Procedure

The search for a route takes place within a three-dimensional corridor according to the tangent process by setting three-dimensional points on the terrain model (Figure 6).

Figure 5. Three-dimensional curve created by the superimposition process

Figure 6. 3D tangent method
The relocatable interpolation points $T_i, T_i \in \mathbb{R}^3$ are set, taking into account the restrictions in the surrounding area. The vertical curve of the tangent intersection points takes place in real time based on the set radius figures $R_k, k = 1(1)m$. A three-dimensional route is created as the result of the initial route search, and the sequence of elements is illustrated using a curvature graph on the horizontal projection and using the course of the gradient on the vertical projection. By shifting the tangent interpolation points $T_i, i = 1(1)n$ in the area $(x, y, z)$, the course of curvature and gradient is completely changed. But if only the height $z$ is corrected, there is just a change in the course of the gradient (embankment cutting change).

In addition to manipulating the intersection points, it is also possible to shift the manipulation points $Q_i = l(1)$. Then the start and finish of the vertical curve are corrected on the horizontal projection.

By using various iteration stages, it is possible to adapt the course of three-dimensional curves to the local terrain and ensure that the design conforms to the guidelines.

4.3 Checking the alignment

- **Design parameters**
  As a result of the rerouting work, the design elements on the horizontal and vertical projections are now clear. Using the curvature graph and the gradient, the individual elements and the sequences of elements can be checked to see whether they conform to the guidelines. Any violation or shortfall of the maximum or minimum figures for individual parameters and in the ratios of adjacent individual elements can be established and documented for particular sections.

- **Accident black spots**
  Based on the accident type maps for the section of the route, the accidents are categorized for the two accident types: driving accidents and accidents in parallel traffic, and they are assigned to the relevant sections of the route. Traffic observations are also made at the localized accident sites.

- **Virtual driving sessions**
  The rerouted virtual road is amended on the basis of video journeys around the elements dominating the driving area (markings, horizontal and vertical restraining elements, traffic signs etc.) and the surrounding elements (trees, bushes, buildings and other building features) and is transferred to the driving simulator. Using a team of test persons, virtual driving now takes place along the existing route. Feature graphs documenting the speed curve, road position and steering wheel movements are automatically drawn up on the basis of the recorded data and the associated quantitative parameters (distribution of the average speed, lane position errors, change of angle errors) are determined for the later relative comparison with the replanning options.

- **Replanning areas**
  By superimposing the shortfall areas in the alignment with the localized accident black spots and the problems in driving behavior established through the virtual driving sessions, the shortfall areas for the replanning process are clearly evident.

4.4 Replanning process

The HighwayDesigner software tool is used for the replanning process. The alignment of the existing road is improved section by section using various iteration stages. The options that emerge are “driven along” virtually on the driving simulator. If the driving behavior is homogeneous after an assessment of the individual parameters and the associated quantitative parameters and there are still deviations in selected individual elements and sequences of elements from the standard rules, the option is still approved as a safe and feasible solution.
4.5 Assessing the driving behavior

4.5.1 Summary

To assess the driving behavior as part of the iterative routing process, virtual journeys take place by means of driving simulation. The driving simulation can basically be performed with 2 different methods:

- Driving simulation using different simulator types (workstation simulator, static vehicle simulator, movement simulator)
- Mathematical simulation and automatic driving using a special driver model (definition of driver characteristics)

A static vehicle simulator was used for the virtual journeys, i.e. driving along the existing and replanning options took place using a team of test persons in line with a precisely coordinated scenario. The following parameters – speed, lane position and steering wheel movements – were selected to assess the driving behavior; this produced the associated driving behavior graphs as average figures and they were compared to the geometric feature graphs (gradient and curvature graphs).

No use has been made of virtual journeys with mathematical simulation for the time being, as the available driver models did not provide any usable results.

4.5.2 Quantitative assessment parameters

The following quantitative assessment parameters were derived for the individual parameters based on the driving behavior graphs to objectively assess the driving behavior:

Speed parameter:

The average speed $\overline{v}$ is created as an average figure for the individual speeds in the relevant cross sections and it illustrates the speed level along the complete section of the route. The associated empirical standard deviation $\sigma_v$ illustrates the deviation from the average speed and is therefore a measurement for assessing the homogeneity of the route characteristics. A small value for $\sigma_v$ expresses a road with a balanced alignment with low speed differences between the individual elements.

The cumulative speed change $k \Delta v$ as a specific parameter for the complete route also helps describe the homogeneity in the alignment in quantitative terms.

- Average speed $\overline{v}$
  \[
  \overline{v} = \frac{1}{n} \sum_{i=1}^{n} v_i = \frac{v_1 + v_2 + \ldots + v_n}{n}, \quad \text{[km/h]} \quad (9)
  \]

- Standard deviation $\sigma_v$
  \[
  \sigma_v = \sqrt{\frac{1}{n} \sum_{i=1}^{n} (v_i - \overline{v})^2}, \quad \text{[km/h]} \quad (10)
  \]

- Cumulative speed change $k \Delta v$
  \[
  k \Delta v = \sum_{i=1}^{n} \Delta v_i = \sum_{i=1}^{n} |v_{i+1} - v_i|, \quad \text{[km/h]} \quad (11)
  \]

Lane deviation parameter:

- Lane errors

Figure 7 illustrates how to determine the lane error SF as a cumulative parameter for all the deviations in contrast with the computed lane tolerance. If a lane error occurs, the vehicle is located on the road’s opposite lane along individual sections and this poses a risk to safety. Sections of the route with a high figure for the cumulative lane error normally illustrate route characteristics that are anything but homogeneous.
Figure 7. Model for the lane error assessment factors

\[ SF = \sum_{i=1}^{n} |SAB_i - ST|, \]  
\[ \text{[m]} \]  \hspace{1cm} (12)

with

\[ ST = \frac{B_{PS} - B_{FZS}}{2}, \]  
\[ \text{[m]} \]  \hspace{1cm} (13)

\[ SAB_i : \text{Lane deviation } i \text{ [m]} \]
\[ ST : \text{Lane tolerance [m]} \]
\[ B_{PS} : \text{Width of road [m]} \]
\[ B_{FZS} : \text{Width of vehicle [m]} \]

Steering angle parameter:

When driving along a road, the driver steers the vehicle along its own lane by moving the steering wheel. The necessary changes in the steering angle resulting from the geometrical change of direction can be determined using the direction angle \( \tau \), i.e. the cumulative geometrical change in the direction angle \( \sum_{i=1}^{n} |\tau_i| \) and the cumulative change in the steering angle \( \sum_{i=1}^{n} |\Delta LW_i| \) have similar values in an ideal situation and the steering angle error \( LWF \) is low. In practical driving situations, however, changes to the steering angle take place as a result of slight movements of the steering wheel without any movement of the wheels (play in the steering wheel). If we neglect this disruptive variable through validation, the figure for the steering angle error is also a quantitative expression for alignment that is anything but homogeneous, i.e. the driver must perform many more movements of the steering wheel with alignment that is not balanced than on a route with a balanced alignment.

- Cumulative steering angle change \( \Delta LW \)

\[ \Delta LW = \sum_{i=1}^{n} |\Delta LW_i| = \sum_{i=1}^{n} |LW_{i+1} - LW_i|, \]  
\[ \text{[degrees]} \]  \hspace{1cm} (14)

\[ LW_i : \text{Steering angle at point } i \text{ [degrees]} \]

- Steering angle error \( LWF \)

\[ LWF = \sum_{i=1}^{n} |\Delta LW_i| - \sum_{i=1}^{n} |\tau_i|, \]  
\[ \text{[degrees]} \]  \hspace{1cm} (15)
\( \Delta L W_i \): Cumulative steering angle change [degree]
\( \tau_i \): Geometric direction angle [degree]

To assess the driving behavior quantitatively, a relative comparison between the assessment factors on the existing road and the replanning options was the only work that was performed initially. Validation involving the establishment of maximum and minimum figures still has to be completed in subsequent experiments.

5. PRACTICAL EXAMPLE

5.1 Details of the task

The developed methodology was tested on a stretch of rural road measuring 10 km. About 60 driving accidents and accidents involving parallel traffic took place here during the period 2011 – 2013 and they were due to wrong driving behavior. The results obtained are explained subsequently on the important section measuring 3 km.

5.2 Checking the existing road

Shortfalls in the individual elements and the sequence of elements on the horizontal and vertical projections emerged from the alignment. By superimposing these shortfall areas with the major elements in the accidents, it was possible to identify 2 areas with a high risk potential (8).

The analysis of the driving behavior graphs showed clearly that there are problems in driving behavior in these areas too. This qualitative statement is underpinned by the figures for the quantitative assessment parameters (Table 1). As a result, areas 1 and 2 are the important replanning areas resulting from the superimposition process.

<table>
<thead>
<tr>
<th>Assessment parameter</th>
<th>Formula sign</th>
<th>Unit of measurement</th>
<th>Existing route</th>
<th>Replanned route</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average speed</td>
<td>( \bar{V} )</td>
<td>km/h</td>
<td>97.92</td>
<td>112.61</td>
</tr>
<tr>
<td>Standard deviation</td>
<td>( \sigma_\tau )</td>
<td>km/h</td>
<td>20.55</td>
<td>3.94</td>
</tr>
<tr>
<td>Lane error</td>
<td>SF</td>
<td>m</td>
<td>6,188.50</td>
<td>0.00</td>
</tr>
<tr>
<td>Steering angle error</td>
<td>LWF</td>
<td>degree</td>
<td>1,281.18</td>
<td>212.34</td>
</tr>
</tbody>
</table>

5.3 Assessing the replanning options

Two options were selected for the comparative experiments. Both options have the same gradient course. The axis in option 1 has an envelope curve as the geometric design between points 0+500 and 1+600 and does not match the design regulations in Germany in this section. Option 2, on the other hand, fully complies with the regulations on the horizontal projection (Figure 9). Both options lead to a significant improvement in driving behavior in comparison with the existing route, i.e. the speed curve, the lane deviation and the steering angle change are all significantly harmonized. This is clear from the relative changes in the figures for the quantifiable parameters in Table 1. Because of the harmonization of the alignment on the horizontal projection (axis), the average speed \( \bar{V} \) increases by about 15% and the deviation in the speed factors \( \sigma_\tau \) declines markedly. The lane error SF for both options is 0, as the vehicles generally remain in their own lane. The steering angle error also declines significantly. The comparison with the replanned options basically shows that the driving behavior is harmonious between 0+500 and 1+600, despite an axis at the localized points area that does not meet the regulations, i.e. even replanning option 1 is feasible in practice.
6. THE RESULTS AND PROSPECTS

The results presented here have clearly demonstrated that the driving behavior can be reliably assessed during the design process by means of virtual driving sessions.

A relative comparison between the existing and upgraded road is possible with the help of the assessment parameters developed here. Maximum or minimum figures for the individual assessment parameters still have to be determined as part of a validation process.

If individual design parameters deviate from the standard values, the driving behavior graphs and the associated quantitative assessment parameters must be drawn on as the professional basis for approving any exceptions.

The new kind of methodology developed here forms a solid foundation for developing a standard procedure for the design process for modification and upgrading work.

Experiments on additional modification and upgrading projects are still required to further develop, validate and introduce this scheme into design practices.

Further research project work will check whether the results of the experiments can be transferred to workplace simulators or whether virtual journeys based on mathematical simulation programs can also provide usable results.
Figure 8. Analysis of the existing road
Figure 9. Replanning options with associated feature graphs
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An Exploratory Study Using Big Data for Improved Safety and Operational Efficiency: A New Zealand Case Study

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MOTIVATION

Concerns related to crashes has increased globally over the years because of the potential level of injury severity sustained by those involved and their resulting economic impacts. “This is especially true for New Zealand with over 4.9 million inhabitants and up to 3.68 million visitors each year (Stats New Zealand, 2017). The countries limited road network of 11,000 kilometers of state highways and 83,000 kilometers of local roads provides a challenge to overseas drivers that must navigate mountain passes to shared heavy light motorways (at different speeds) (Walker, 2014). Recently New Zealand has joined a long list of jurisdictions that are establishing Vision Zero initiatives and directing policy makers towards the implementation of these initiatives (Office of the Associate Minister of Transport, 2018). To achieve this goal, New Zealand is focusing on mechanisms to effect change that include changing government policy, setting sensible speed limits, reducing the tolerance for speeding during periods of high travel such as public holidays, media campaigns, and better road engineering. This is important because in 2017 there were 379 deaths on New Zealand roads (NZ Ministry of Transport, Research and Statistics, 2017a). Along with the all too high human cost, New Zealand's Ministry of Transport estimates the social cost of road crashes and associated injuries in 2016 to be NZS4.17 billion (NZ Ministry of Transport, Research and Statistics, 2017a). This is estimated by the amount of money that the New Zealand population would be willing to pay for safety improvements that result in the expected avoidance of one premature death. These costs will only continue to grow if nothing is done to help mitigate and reduce roadway crashes.

In New Zealand the major causes of road death on open roads are losing control, travelling too fast for the conditions, driving impaired, and failing to keep left. In regard to Urban causes, failure to give way and failure to see the other party lead the way (NZ Ministry of Transport, Research and Statistics, 2018). However, in recent years there has been a reduction in the average vehicle speed across the New Zealand road network, and the incidence of vehicles travelling over the speed limit has also seen a reduction. An analysis by Sam Warburton (2017) looked at crashes per kilometer driven in New Zealand and found that the chance of a person in a car dying on the road was 41 percent higher than it was in 2013, and 12 percent higher than 2016. Moreover, New Zealand is a popular tourist destination, with 5.6% of GDP being generated from tourism (New Zealand. Statistics New Zealand., 2016). According to the New Zealand Ministry of Transport, 4.3 % of fatal and injury crashes occurred when an overseas driver was at fault. Most of these crashes were the fault of drivers that originated from countries that also drive on the left side of the road, indicating that unfamiliarity with left hand driving was not to blame (NZ Ministry of Transport, Research and Statistics, 2017b).

With these statistics in mind, the goal of this paper is to better understand contributing factors that increase an individual’s road risk. The road is a dynamic system, evolving throughout the day as the population uses it. As such the risk of an individual road can therefore be defined in this study by how fatigued, frustrated, and/or familiar the users of that road are at the time they are driving. To accomplish this, this paper utilizes telematics data collected in New Zealand that provides a unique opportunity to study human factors related to driving at an aggregated level and provides insights into how the collective population affects road risk across the day. This will be done through an exploratory statistical analysis of that telematics data and the development of risk measures for fatigue, frustration and familiarity of the road network. This data is collected in collaboration with EROAD, that have a strong presence in New Zealand, and have the critical mass of telematics devices on the roads to study this behavior and provide an additional level of risk that can be used to enhance the existing models to include population behavior. Because of the regulatory motivations behind the installation of these devices, the accuracy and polling frequency are higher than comparable sources, enabling the analyst to better understand the network corner by corner. The use of
telematic data to study driving behavior is not new (Brackstone et al., 1999; Hickman and Hanowski, 2011; Wahlstrom et al., 2015; Wouters and Bos, 2000), however past studies have been limited by the number and quality of the data collection process. In addition, these same studies have either been cell phone based and/or instrumented vehicles (Botzer et al., 2017; Hickman and Hanowski, 2011; Itoh, 2008). The current data set used for this study is collected directly from various vehicle classes in the New Zealand and amount to a sample size of over 1 billion records (See Data Description). To the best of the authors knowledge these are the first attempts at utilizing such data to better understand fatigue, frustration and familiarity of the road network.

The following sections outline the data, provide an exploratory statistical analysis of the data to gain insights into the contributing factors related to fatigue, frustration and familiarity.

DATA DESCRIPTION

The data used for this study was generated from a GPS-GNSS enabled device installed in vehicles owned by customers of EROAD, a global regulatory telematics company. The devices are installed in a wide variety of vehicle makes (over 2800) and across 39 industry sectors. Specifically, the data was collected from 20,866 LIGHT vehicles (GVM<3,500kg) and 32,740 HEAVY vehicles (GVM>3,500kg).

The device central collects GPS and telematics data continuously, which is transmitted to a central platform approximately every 250 meters when the vehicle is in motion, and for significant events such as harsh braking, ignition on/off, and idling. The events are enriched with information about the road, obtained from HERE Maps (a map generating company). For example, the posted speed limit was abstracted, which facilitated the calculation of vehicle speed (e.g., if they were speeding), and the road functional class as described below.

Speeding events were defined when a vehicle exceeded the speed limit by 5 km/h or more. Heavy vehicles have a reduced speed limit of 90 km/h in an otherwise 100 km/h limit, therefore speeding events were counted when a HEAVY vehicle exceeded 95 km/h. To satisfy privacy concerns, none of the speeding events had information that could be used to identify the EROAD customer.

Harsh braking events were generated on the device when the change in velocity exceeds 10 km/h in 1 second. This equates to a deceleration of 2.77ms² or 0.28g.

The functional class of the road was used to summarize the results throughout the paper and ranges from class 1 roads designed for high speed, high volume traffic to class 5 roads designed as local access. As seen in Figure 1 the density of these classes increases to serve smaller regions. The road network reference being used is divided into road segments, primarily at intersections, but also at jurisdictional boundaries.

Finally, a trip is defined as the duration between a vehicle’s ignition on event and ignition off event. Outside of the scope of this study was trip chaining, whereby we would expect longer travel times when combining trips with short intervals between them, for example courier drivers delivering packages and turning the vehicle off for just a few minutes, or vehicles waiting at rail crossings and not idling.

Table 1. Description of the functional road classes used in the paper

<table>
<thead>
<tr>
<th>Road Class</th>
<th>Description</th>
<th>Count of Road Segments</th>
<th>AADT</th>
<th>Speed Limit Range (km/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>National High Volume</td>
<td>19,354</td>
<td>&gt; 35,000</td>
<td>70 - 110</td>
</tr>
<tr>
<td>2</td>
<td>Regional, linking remote regions</td>
<td>11,811</td>
<td>&gt; 15,000</td>
<td>60 - 100</td>
</tr>
<tr>
<td>3</td>
<td>Arterial, critical connectivity</td>
<td>32,462</td>
<td>&gt; 5,000</td>
<td>50 - 100</td>
</tr>
<tr>
<td>4</td>
<td>Collector, linking between 200 and 2000 populations</td>
<td>42,420</td>
<td>&gt; 3,000</td>
<td>50 - 100</td>
</tr>
</tbody>
</table>
As part of the investigation into how frustration is affected by the curvature of the road, a curvature ratio is defined. This curvature ratio is defined as the ratio of the straight-line distance between either end to the contoured length (see Figure 2 and 3). Accounting for compound curves, reverse horizontal curves, or changes in vertical alignment was out of the scope of this study.

Figure 1. Map of Road Functional Classes. Class 1 (state highways) at the left to Class 5 (local roads) at the right

Figure 2. Calculation of curvature ratio, dividing the shortest distance by the contoured distance.
RESULTS

Below we describe the analysis in terms of three risk measures for fatigue, frustration, and familiarity. For fatigue we looked how distant each road segment is from the origin of the vehicles that visit it. Frustration was measured by analyzing how driver's propensity to speed changes based on the curvature of the road preceding their current location. Finally, for familiarity we looked at the proximity of speed and harsh braking events to the start and stop location of a trip.

Fatigue

Time to Arrive

According to the New Zealand Ministry of Transport, in 2016 fatigue was identified as a contributing factor in 28 fatal crashes, 119 serious injury crashes and 438 minor injury crashes. These crashes resulted in 36 deaths, 160 serious injuries and 574 minor injuries. The total social cost of crashes involving driver fatigue was about $291 million; this is about 7 percent of the social cost associated with all injury crashes (Ministry of Transport, Research and Statistics, 2017c).

For this study access to individual driver's work plans or rosters were not obtained, so instead an estimate of fatigue based on trip time was derived. Over 9.5 million trips were studied where vehicles visited a total of 49.2 million road segments. The trips were then filtered to remove those where the amount of rest time before the trip was less than one hour. The median time it took each vehicle to reach each road segment was then calculated and this was split by the vehicle weight type.

This allowed us to show a distribution of travel times from the last sufficient rest period to the current road segment. This distribution is shown in Figure 4 below, which plots the distribution of median travel times for each road segment. We can see that HEAVY vehicles visit road segments after driving for significantly longer than LIGHT vehicles, which fits with the assumption of HEAVY vehicles undertaking long haul duties, and LIGHT vehicles being used for localized travel.
In order to create an index that represents the remoteness of the road from the drivers rest location, the median travel time between the top and bottom 5% of all travel times was normalized, and the visit counts to the segment were normalized between the top and bottom 5% of all segments. These two indexes were in the range 0 - 1, and were combined into a single index using equation 1,

\[
\sqrt{\frac{(tti^2 + vci^2)}{2}}
\]  

(1)

Where, tti is the travel time index and vci is the visit count index. Because the trips were split into those from HEAVY and LIGHT vehicles, the final step was to calculate the combined index using a weighted average between the HEAVY vehicle index and LIGHT vehicle index, the visit count was used as the weighting variable.

The road segments were split by this final index using kmeans into 5 clusters (Lloyd, 1982). Figure 5 below shows the extremes of these clusters, with the lowest risk roads on the left, located mainly in city centers, and the highest risk roads on the right. The intermediate clusters (not shown) are located on a mixture of arterial and collector roads. We can see that the riskiest roads are visually close to functional class 1 roads, which makes sense as the travel time to reach these roads is a lot higher. However, we also see that roads in urban areas score high on this index.
Frustration

Curvature

New Zealand has some of the most challenging roads in the world, particularly when it comes to travel in rural areas. The topology of the land, and the layout of our major centers means that many of the functional class 1 and 2 roads, designed for high volume, high speed travel must be designed with a significant number of curves.

We describe each road segment in terms of its curvature and the effect that has on the propensity to speed. To understand this behavior we looked at each location event that was preceded by either a cornering event or another location event. Cornering events are generated when a vehicle has completed a bearing change of at least 35 degrees. With these two scenarios, corner-straight (cs) or straight-straight (ss) we plotted (Figure 6) the distribution of speeds that fell within +/- 10% of the road speed limit. We also ensured that the preceding event was on a road with the same speed limit as the subject event. The distance between the subject event and the preceding event was 250m +/- 13m. We eliminated events where the driver was seen to change their speed by more than 10km/h per second, which would have generated a harsh acceleration or harsh braking event.
We see in the graph above the two distributions for each event tuple (corner-straight and straight-straight). In all speed limits we see a greater proportion of speeds under the speed limit in corner-straight tuples. This indicates that drivers are accelerating out of corners but are more likely to remain under the speed limit. In many roads this could be because of the lead up to another corner, where exceeding the speed limit will likely result in a sharp deceleration to meeting the next corner's advisory speed. In the straight-straight event tuple we see a larger proportion of events that exceed the speed limit than in the corner-straight event tuple. Even though the inter-event distance is 250m +/- 13m, which is sufficient to accelerate to above the speed limit from a corner, drivers are more likely to exceed the limit when they are already travelling in a straight road. This speed violation is less likely however on roads with lower speed limits, perhaps due to increased congestion and less opportunity to speed.
For HEAVY vehicles we see the largest change in speeding behavior on functional class 1 roads, where vehicles exhibit 30.5% more over-speeding in straight-straight tuples compared to corner-straight tuples. Table 2.

Table 2. Proportion of HEAVY vehicle over-speeding when following a curved or straight road

<table>
<thead>
<tr>
<th>Road Class</th>
<th>Proportion of Overspeed in Corner to Straight Event Tuple</th>
<th>Proportion of Overspeed in Straight to Straight Event Tuple</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. National</td>
<td>0.323</td>
<td>0.422</td>
</tr>
<tr>
<td>2. Regional</td>
<td>0.282</td>
<td>0.364</td>
</tr>
<tr>
<td>3. Arterial</td>
<td>0.273</td>
<td>0.333</td>
</tr>
<tr>
<td>4. Collector</td>
<td>0.270</td>
<td>0.346</td>
</tr>
<tr>
<td>5. Access</td>
<td>0.256</td>
<td>0.326</td>
</tr>
</tbody>
</table>

Figure 7. Proportion of overspeed events from HEAVY vehicles that follow a straight road or curve, split by road class. Proportions are of all events between -10% and +10% of speed limit.
For LIGHT vehicles we see a marginally higher incidence of speeding when exiting a corner on functional class 5 (4.1%), class 4 (1.1%) and class 3 roads (2.1%). This could be due to the faster acceleration profiles of LIGHT vehicles on local roads. As with the HEAVY vehicle data, we also see an increase in the propensity to speed in functional class 1 roads (10.8%) roads when vehicles are already travelling on a straight road. This could be due to overtaking lanes. The results are presented in Figure 8.

Table 3. Proportion of LIGHT vehicle over-speeding when following a curved or straight road

<table>
<thead>
<tr>
<th>Road Class</th>
<th>Proportion of Overspeed in Corner to Straight Event Tuple</th>
<th>Proportion of Overspeed in Straight to Straight Event Tuple</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. National</td>
<td>0.295</td>
<td>0.327</td>
</tr>
<tr>
<td>2. Regional</td>
<td>0.285</td>
<td>0.299</td>
</tr>
<tr>
<td>3. Arterial</td>
<td>0.309</td>
<td>0.302</td>
</tr>
<tr>
<td>4. Collector</td>
<td>0.354</td>
<td>0.350</td>
</tr>
<tr>
<td>5. Access</td>
<td>0.323</td>
<td>0.310</td>
</tr>
</tbody>
</table>
To translate this back to an individual roads likelihood of facilitating speeding events, we use equation 2 on each road segment.

\[ c \text{ is the curvature of the road as defined above, } s \text{ is the reverse of } c \text{ indicating the amount of straightness, } PLSS \text{ & } PHSS \text{ are the proportions of overspeed for LIGHT & HEAVY vehicles respectively in a straight-straight tuple, } PLCS \text{ & } PHCS \text{ are the proportions of overspeed for LIGHT & HEAVY vehicles in a corner-corner tuple.}\]

This produced a range of scores in the domain -1.369 to 10.439. We truncated this distribution to the upper and lower 1%, and coerced values that lay outside to these limits. Finally, this domain of -1.369 to 8.429 was normalized to the range 0 to 1 by equation 3.

\[ \log(0.1^s \times PLSS + c \times PLCS + 8.429 - 1)/0.1 \text{ (3)}\]

where \( x \) was the score and \( p01 \) was the 1% percentile of the score, and \( p99 \) was the 99% percentile of the score.
Familiarity

Proximity of harsh driving to destination

It is widely accepted that crashes are more likely to occur when a vehicle is close to their home location. This was studied by Burdett et al., (2017). The study used the New Zealand Household Travel Survey and found that roads within 11 km (6.8 miles) of home accounted for half of all travel and 62% of all crashes. The reason behind many of these close-to-home crashes involve a lapse in attention. The study also suggests that local roads are over represented in the dataset.

For this study, we also looked at the proximity of harsh braking events to the final destination of their respective trip. The difference in approach to the Burdett et al., (2017) paper is that we are able to focus on harsh braking and speeding as precursors to a crash. The authors measured the straight line distance between the crash and the center of the driver’s home suburb, we are measuring the exact distance from the location of the poor driving event, and the final destination of the driver. Given the vast difference in the shape of suburbs, this distance is much more accurate. Lastly, we are able to compare the results of harsh driving to destination distances where that final destination is a common location for the driver vs any location.

We studied a sample of 1.9 million trips that showed 610,000 harsh braking events. Of these trips, 71.32% terminated on local roads (functional class 5), and 48.26% of harsh braking occurred on these roads. On the other functional classes, harsh braking is more represented than trip terminations, indicating this behavior happens during a trip.

We calculated the distance between the location of the harsh braking event and the final destination and normalized this over the trip distance. We found that 50% of harsh braking events occurred within 8.6km of the trip's final destination. Furthermore 90.4% of harsh braking events occurred in the last 50% of their trip. When looking at the distribution of event to stopped location distances, normalized to the trip distance, we find that fewer harsh braking events occur at the start of the trip, and more occur at the end of the trip.

We saw no relationship between the intensity of the harsh braking event and the proximity to the origin or destination.
The same 1.9 million trips were also analyzed for speed events. When normalized across the trip, the distribution of speeds to the start and stop position is different to that of harsh braking. We see that most speeding occurs in the middle of the trip, after a vehicle is approximately 10% of the way through its trip. Speeding again drops off as the vehicle approaches its destination. This is understandable given the large number of trips that terminate on functional class 5 roads with lower speed limits, however we have seen in the curvature analysis that there are pockets of speeding occurring after corners on these road classes.

To create the index that represents the propensity to speed when near the final destination we first describe each road segment by the average location along the length of all trips that encounter it. The density curves above were used to obtain the probability of speeding or harsh braking for a given location along the trip. This was repeated for HEAVY and LIGHT vehicles for both harsh braking and speeding curves. The result is that road segments that are consistently close to the destination of the trips that intersect them will show a high-risk of harsh braking, and a lower risk of speeding. Conversely, segments that are close to the origin of the trips that intersect them will show a lower likelihood of both harsh braking and speeding. The
four resulting indexes where then simply normalized between the lower and upper 1% of values, and the mean was taken to reduce into a single familiarity index.

Combining Indexes together

The last part of the analysis was to combine the three indexes for frustration, familiarity, and fatigue into a single index that can be used to make routing decisions on. We chose to simply take the mean in this case. The resulting distribution shows that functional class 5 roads show a consistently lower risk index than functional class 1. Road classes 1, 2, and 3 show a multi-modal distribution which is driven by varying speed limits, whereas class 4 and 5 speed limits are more consistent.

Mapping routes

The resulting index was split into 5 clusters using kmeans and plotted on a map. Figure 13 shows an example of a route choice between Taupo and Rotorua. The left-hand route is 107km, whereas the right-hand route is 81.4km according to Google. We calculated the cumulative risk index for each route by taking the weighted average of the risk indexes for each road segment, weighted by the road segment distance. This is because this current model cannot assess where in the road segment the vehicle is subjected to the calculated risk, and the lengths of the road segments vary substantially.

Of the two route choices in figure 13, the left-hand route shows a weighted mean index of 0.408 whereas the right hand route shows a weighted mean index of 0.595. Thus, the right hand route has 23.9% less distance, but 45.8% higher risk.
**SUMMARY AND CONCLUSIONS**

This paper set out to describe the road network in terms of the risk imposed on individual drivers by others using the network. We explored ideas such as changing shift schedules as a measure of fatigue, the number of times a driver visited a particular road segment as a measure of familiarity, and the driver’s adherence to the speed limit on the various roads they encounter across their trips. We settled on three metrics that demonstrated a strong correlation with harsh braking and speeding, and attempted to convert these measures into a single metric that could be used to make improved routing decisions on the network. This example clearly shows that the shortest route, that would normally be suggested by most consumer routing applications, exposes the vehicle to a higher risk than the alternative route.

Optimized routing offers a competitive advantage for many transport operators, as margins are waning, they are looking for more ways to increase efficiency, and reduce wear and tear on the vehicle. Consumer grade routing applications work to minimize a distance-based penalty between an origin and destination. Some more advanced algorithms will also include other factors such as topology, intersections, congestion, and special use roads. We believe this is the first implementation that offers a population risk penalty to also optimize for. As a next step we will be implementing this algorithm over a directed graph of road segments and exploring how the risk model changes across the time of day or day of week. We will also automate the calculation of the various risk indexes to increase the sample size from one billion records to the full EROAD dataset.

The potential for EROAD customers is that we can dynamically route their vehicles as we see a change in the network ahead of their trip and offer drivers a safer alternative to reach their destination on time. Having an optimized route model also offers EROAD customer’s a competitive advantage in terms of their obligations under the Health and Safety Act 2015. Moreover, we believe we are in a position to better inform the design of static factors such as the design of infrastructure and dynamic factors such as choice of route, and signage to soften the negative consequences of situational factors.
EROAD’s mission is to solve complex transportation problems and the goal of this research project was to showcase options that assist in achieving Vision Zero. It will further give government agencies a new layer of information for evidence-based spending on infrastructure funding.

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Toward a Sustainable Road Safety in Kuwait: The Role of Driver Behavior

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ABSTRACT

Traffic congestion and accidents are one of the challenges facing Kuwaiti government on a daily basis. Traffic congestion on Kuwait road network causes considerable delay for all road users, which negatively impact their driving behavior. It has also severe impacts on both country economy and quality of life. Due to this fact, an assessment is made for any new or adjusted part of the traffic system. Taking into consideration the data available about both the system and the users will insure an adequate design. However, in the case of Kuwait and based on data availability, one factor was considered that could affect or change the outcome of the analysis, which is the behavior of road user (i.e. the driver). Driver’s behavior proved to be an effective variable and could be predicted if a pattern is established. In this paper, the driver’s behavior was monitored especially lane changing activities in two merging and diverging sections. This was done via video recording to get a connection between traffic parameters and the drivers’ behavior. This will create the necessary awareness towards the driver behavior impact on traffic accidents and the number of conflict points. Different scenarios were implemented using the model created in Synchro to scrutinize the change in the number of conflict points and type of accidents. In this case, a 7% increase in the conflict points of Estiqlaal Street and a 17.55% increase in Fahaheel road were witnessed due to the sole effect of driver behavior without taking the change in layout into consideration.

Keywords: Traffic; Congestion; Kuwait; Driver behavior; Conflict points; Safety
1. Introduction

Before going through this paper, a small definition was presented to give a better understanding of this subject since it is easy to get confused by the terminologies. The deference between the weaving section and merging and diverging sections is that a weaving section considers a part of the freeway that has an in way or an entrance to another road, which is followed by an exit. On the other hand, merging usually occurs when two traffic streams merge into a one stream, which can also occur in ramps while diverging being almost the opposite; streams are isolated into two separate traffic streams. Previous studies developed some hypothesis regarding the ramp metering and bottlenecks and compared them with the empirical formula. They found that it is possible to increase the bottleneck limit by delaying, and sometimes removing the bottleneck initiation to serve more traffic (Zhang and Levinson, 2003). Others tried a new approach, which included a size to capacity relation to get the level of service (LOS) of the segment. The traffic quality (LOS) can be gotten straight forward on the freeway, on the ramp entrance or the exit as indicated by Wua and Lemkeb (2014). They mentioned that their developed model has a uniform function for all merging, diverging and weaving sections and it is easy to calibrate. In order to develop a more innovative way to study the ramps in the motorway, a ramp metering and hard shoulder strategies had been installed (Haj-Salem et al, 2014). After the observation, the result was good indicating that when the hard shoulder was closed the ramp metering had increased the speed by 11%. The overall time consumed in the network had decreased by 4.4% and the total number of kilometers by 6.3%. On the other hand, when the hard shoulder was in the open position the capacity increased, which increased the mean speed by 12% (Haj-Salem et al, 2014). Kusuma et al. (2014) presented two empirical data collection and data extraction process, which they collected from surveillance cameras and loop detection measurements, to analyze the drivers’ behavior in a weaving section. The study had successfully identified some of the driver’s behaviors indicating that 25.25% of the
weaving movements happen in the initial 50 – 100 m of the merging section. Also, it showed that 30% of the entire traffic was associated with the one path changing movement transit time for the lane change, which was 4.09 seconds. Also, the auxiliary lane had delayed the lane change movement, which affected the traffic performance. In Taiwan, a study was done on the driver’s behavior at a new weaving section. The aim of the study was to get a relationship between the traffic parameters of speed, density and capacity and the behavioral part focused on the lane changing and weaving activities. The results were compared to the highway capacity manual. It was found that the speed and the flow rate were within the limits while the density was higher than the HCM values (TsaiP and Cho 2005). Hansell (1975) conducted a study to evaluate the operational performance and accident’ risk at four different bridges with collector-distributor streets and three interchanges without collector distributor streets. Results showed that the collector-distributor road was effective in improving traffic operation and minimizing weaving effect. Stefaniak and Wallace (1987) conducted a study to evaluate the collector-distributor road along a 16-mile segment between Washington D.C. and Frederick, MD. The study used the operational data of the collector-distributor road. The aftereffects of the examination suggested a 12-foot path width for collector-distributor streets with 2 or 3 paths and a 15-foot path width for collector-distributor comprising of just a single path. The least shoulder widths were suggested as 4-ft left and 14-ft right with a total minimum of 4 feet. Ward (1990) proposed an improved type of arterial streets (Strategic Arterial Street System) and conducted a study to evaluate the proposed arterial. The proposed geometric design included partial control of access, median divided roadway, no left turn, and 40 to 50 mph design speed. Geedipally et al. (2014) examined the effect of ramps’ geometry on crashes and their severity. The study indicated that the barrier presence, the number of complete paths, zone category, and ramp type had a large impact on the quantity of high-severity accidents on ramp sections. Brown and Tarko (1999) established a binomial relapse model to assess the number
of crashes on an urban multilane arterial segment. Characteristics of access control, geometry, and number of accidents were used in developing the estimation model. The developed model was used to expect the whole amount of accidents, number of property damage, and number of deaths and injury accidents. Results showed that the access controller had a useful impact on safety. Harwood et al. (1986) estimated the safety, operational, and cost characteristics associated with different multilane designs for suburban highways. For each design alternative, the crash data, the advantages and disadvantages were presented. Created on many serious influences and current condition, an appropriate design alternative was selected to be used on the suburban highway. Harwood et al. (1997) performed a study to improve the highway security manual (HSM) to forecast highway safety act in town and residential zones. The study presented several methodologies to predict traffic accidents in urban and suburban areas. Datta et al. (1982) inspected the town major traffic crashes to classify the essentials to decrease the amount of crashes. In this investigation, roadway sections situated in 19 metropolitan zones were examined. The outcome incorporated a few rules to distinguish crashes issues and select better safety countermeasures. Gipps (1986) formed a lane altering choice model to be utilized in tiny road traffic simulator. His model covered traffic signals, obstacles and heavy vehicles affecting the driver’s decision when changing the lane. However, some factors affecting the driver’s decision were not considered such as inconsistency and non-homogeneity. Yang and Koutsopoulos (1996) established a path altering model, which is rule based and can only be used in freeways. They classified lane change activity to mandatory lane change (MLC) and discretionary lane change (DLC). Another investigation that took the path change conduct utilizing a direction data index gathered at a weaving segment was done in France by Marczak et al. (2016). The influence of driver behavior on traffic was examined with four sorts of information: driver behavior survey, driving simulation, on-street analysis and realistic driving investigation (Burdett et al., 2016; Cheng et al., 2011; Summala et al., 1996). Another study
showed that even when in-vehicle gadgets are hand-free, important changes in driver performance might happen because of the interruption associated to their use (Kaysi & Abbany, 2007). Many studies considered driver performance as a reason in most accidents (Petridou and Moustaki, 2001). Therefore, it’s so important to identify drivers with dangerous driving habits that increase the possibility in getting involved in accidents. Different procedures had been developed for this task that include statistic profiles (Wundersitz and Hutchinson, 2008), self-reported behavior and danger preferences (Goldenbeld and Van Schagen, 2007) and hazard discernments (Machin and Plint, 2010; Musselwhite, 2006).

2. Problem Statement
Traffic studies can be narrow and systematic especially in Kuwait. The focus of the study is the facility (i.e. network, street, traffic signals) and they may improve or solve a problem for an amount of time. However, the other part and a big part of the network is being neglected and that part is the driver. So, in this study the focus was to find more on the impact of driver’s behavior on the road network.

3. Methodology
The aim of this paper was to study the driver’s behavior at diverge and merge sections and to create a model for the two sites using Synchro and SIM. The developed model provided the conflict points and the environmental aspects for the current situation and after adjustment. To achieve these objectives, the following steps were implemented. First of all, five locations were selected based on the ability to install the speed monitors and cameras to collect the necessary data, which include the type of vehicles, speed, and number of cars entering and leaving the merge and diverge sections. The desired number of locations was five locations to ensure that the data for such type of work is enough. The perfect location for this study was a section with a merge to the main road and a diverge from the main road to a secondary road where each of them (merge and diverge locations) should have enough space to mount the cameras, and to
enable the team to collect the desired data. After choosing the desired locations, three of them were discarded for several reasons; it was hard to install the devices due to approval issues by the Ministry of Interior (MOI). The remaining two sites were already approved since the devices were already installed and the data collection was about to start nearly at the same time the study conducted. Some of the other locations had surveillance cameras installed on the highway. However, after examining the recordings, the data could not be collected from them as the view was very far and some angels of the weaving sections were not covered. Another reason was the low budget since the speed monitors, the video cameras and the manual labor were rented by the hour. Another point was the availability of another count to validate the collected data. Secondly, traffic monitors were installed to identify the peak hours at each location. After confirming the peak hours, the devices were installed in the same day of the following week to make sure that the data was correct. The data collected form the monitor were density at each lane, speed for each vehicle in every lane to obtain the average speed at the peak hour and the type of vehicle so that the model can express the actual users of the network. Thirdly, after identifying the peak hours, the video recording and manual recording were done in these hours for the two locations. Then to validate the data collected; another video recording was done at the same location and the same days the recordings were done. For example, if the recording was done on Wednesday the second recording was done on the next Wednesday at the same hours. Fourthly, the data was analyzed, categorized and sent to Synchro to create models simulating the same conditions at the study locations using the collected data. Models’ behavior inputs were set to default as this how it was used for design in Kuwait. Then, the output was generated and the results included the number of conflict points, the type of possible accidents from these conflict points and Carbon Monoxide (CO) emissions of the original (current) location without any modifications. Finally, after analyzing the models based on the current situation, some adjustments that could enhance the existing
situation were done. These adjustments were applied for both models and the effects of such changes were recorded and compared with the actual model to observe their role in changing traffic and drivers’ behavior. The comparison was done via SimTraffic and Surrogate Safety Assessment (SSAM) to get the conflict points and types of accidents that could happen in both locations. CO emissions of both locations with and without adding the driver’s behavior as a variable were also calculated.

3.1 Proposing Improvement Scenario

To give a better assessment of the current network condition and especially for the ramp section, the current layout was simulated to check for any unusual behavior in the network that would not reflect the actual situation. Proposing the improvements’ scenarios is usually done through engineering experience with a previous similar work. In this paper, several meetings were held with the contractor of the ministry of interior, responsible for designing the road networks, to discuss the existing condition and propose a possible solution. The proposed solution must be evaluated using a simulation program, Synchro. In addition, the safety aspects of the proposed solution must be evaluated using the Surrogate Safety Assessment Model (SSAM). SSAM model analyzes the safety aspects using vehicles trajectories from Synchro. The proposed engineering solution was to add a collector lane to the existing ramp. It was assumed that this solution will provide a breather to the drivers entering the highway and a heads-up to the drivers in the main road.

4. Data Collection and Entry

For Al Istiklal Road, the first site was in Bunaid AL-Gar area. The total length between the exit and the entrance was 271 meters. Figure 1 shows the positions of the video cameras, the manual counters (indicated by the blue line) and the digital counters (indicated by the red line). The digital counter was installed first to get the peak hours. The count was done two times on Wednesday 9/3/2016 and another count was done on the next Wednesday 16/3/2016 to make
sure that the data collected was correct. Then the videos recording was done on 23/3/2016 from 07:00 am to 09:00 am with the manual count, which was positioned to get the best angel for observing the lane change behavior during exiting and entering the freeway. Table 1 shows the vehicle count’s configuration system at Istiklal Road.

Table 1: Vehicle count’s configuration system at Istiklal Road

<table>
<thead>
<tr>
<th>Study section</th>
<th>Length (m)</th>
<th>Number of lanes</th>
<th>Width (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main-line</td>
<td>380</td>
<td>3</td>
<td>9.1</td>
</tr>
<tr>
<td>exit</td>
<td>48</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>Entrance</td>
<td>45</td>
<td>1</td>
<td>4</td>
</tr>
</tbody>
</table>

Figure 1: The Study Area – Istiklal Road

For Al Fahaheel Road, the second site was near Bayan area. So, the movements observed was the ones exiting the freeway and entering the freeway. The position of the video recorder and speed monitors are shown in figure 2. The same procedure in the first site was done in the second site regarding the number of counts and deciding on the peak hours. The count took place at the same days from 12:00 pm to 02:00 pm; the peak hours based on the count conducted in this area. Table 2 shows vehicle count configuration for Fahaheel Road.
Table 2: Vehicle count configuration for Fahaheel Road

<table>
<thead>
<tr>
<th>Study section</th>
<th>Length (m)</th>
<th>Number of lanes</th>
<th>Width (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main-line</td>
<td>720</td>
<td>3</td>
<td>10</td>
</tr>
<tr>
<td>exit</td>
<td>236</td>
<td>1</td>
<td>3.9</td>
</tr>
<tr>
<td>Entrance</td>
<td>325</td>
<td>1</td>
<td>3.9</td>
</tr>
</tbody>
</table>

Figure 2: The Study Area – Fahaheel Road

4.1 Data Requirements

The data needed for this work were speed, density, capacity, lane change activity observed from the video recording and the site geometry, which was used as an input to Synchro to produce a model for the two sites and an analysis of the traffic parameters with the lane change activity being used as a calibration factor for the model. After creating the model, a simulation was done besides an analysis using Simtraffic and Surrogate Safety Assessment (SSAM).

4.2 Data Collection

The data obtained from the manual count are shown in tables 3 and 4 below. Table 3 shows the number of cars entering the inner street in the Istiklal road. This manual count was done at the peak hours, these peak hours were identified by installing the counters for one week to observe
the condition of the road. An interval of 15 minutes was taken to insure that the count was done properly as per the contractor’s instructions and after 15 minutes the counter recorded the number of cars in the note and resets the count.

Table 3: Vehicle Count for Istiklal Road Entering

<table>
<thead>
<tr>
<th>Time</th>
<th>IN</th>
</tr>
</thead>
<tbody>
<tr>
<td>7:00 - 7:15</td>
<td>90</td>
</tr>
<tr>
<td>7:15 - 7:30</td>
<td>90</td>
</tr>
<tr>
<td>7:30 - 7:45</td>
<td>123</td>
</tr>
<tr>
<td>7:45 - 8:00</td>
<td>120</td>
</tr>
<tr>
<td>8:00 - 8:15</td>
<td>106</td>
</tr>
<tr>
<td>8:15 - 8:30</td>
<td>131</td>
</tr>
<tr>
<td>8:30 - 8:45</td>
<td>141</td>
</tr>
<tr>
<td>8:45 - 9:00</td>
<td>103</td>
</tr>
<tr>
<td>9:00 - 9:15</td>
<td>110</td>
</tr>
</tbody>
</table>

Then, the number of cars entering Bunaid Al-Gar and the number of cars exiting the side ramp and joining Istiklal street are shown in table 4 below.

Table 4: Vehicle Count for Istiklal Road Exiting

<table>
<thead>
<tr>
<th>Time</th>
<th>NBT</th>
<th>NBL (Out to Rd. 30)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7:00 - 7:15</td>
<td>55</td>
<td>111</td>
</tr>
<tr>
<td>7:15 - 7:30</td>
<td>65</td>
<td>131</td>
</tr>
<tr>
<td>7:30 - 7:45</td>
<td>63</td>
<td>166</td>
</tr>
<tr>
<td>7:45 - 8:00</td>
<td>57</td>
<td>180</td>
</tr>
<tr>
<td>8:00 - 8:15</td>
<td>78</td>
<td>154</td>
</tr>
<tr>
<td>8:15 - 8:30</td>
<td>79</td>
<td>146</td>
</tr>
<tr>
<td>8:30 - 8:45</td>
<td>76</td>
<td>183</td>
</tr>
<tr>
<td>8:45 - 9:00</td>
<td>77</td>
<td>82</td>
</tr>
</tbody>
</table>

In table 4, the stream of cars was divided into two categories. The first one was the NBT, which indicated the number of vehicles that will continue on the secondary road and will not enter the main road. The other one was NBL. It indicated the number of vehicles that will enter the main road.

A sample of the data collected from the speed monitor for site number one, which classified the speeds into six categories, is shown in table 5.

Table 5: Vehicle Speed Classification for Istiklal Road
To classify Table 5; time column indicates when the counter entered the collected data. SB indicates the south direction in which the vehicles are going out of Kuwait City in the opposite direction of the studied section. NB represents the vehicles going to Kuwait City toward the intended section. Speed was classified into six categories starting from (0-40) km/hr and ending with speeds above (120) km/hr. The same method profile was made for Fahaheel road. Table 6 below, has the same direction classification SB and NB with the information gathered being the count of vehicles based on their lengths. This data was inputted in the model to ensure that the types of vehicles using the road were the same in the model vehicle classification based on length. This was used as an input to synchro to account for trucks, SUVs and buses.

Table 6: Vehicle Count Classification for Fahaheel Road

<table>
<thead>
<tr>
<th>Direction</th>
<th>SB</th>
<th>NB</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Speed Classification</td>
<td>Speed Classification</td>
</tr>
<tr>
<td></td>
<td>SB Total</td>
<td>NB Total</td>
</tr>
<tr>
<td></td>
<td>0-40</td>
<td>40</td>
</tr>
<tr>
<td>Time</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0:00</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>0:15</td>
<td>2</td>
<td>5</td>
</tr>
<tr>
<td>0:30</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0:45</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td>1:00</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>1:15</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>1:30</td>
<td>3</td>
<td>9</td>
</tr>
<tr>
<td>1:45</td>
<td>0</td>
<td>4</td>
</tr>
<tr>
<td>2:00</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2:15</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2:30</td>
<td>0</td>
<td>2</td>
</tr>
<tr>
<td>2:45</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3:00</td>
<td>7</td>
<td>0</td>
</tr>
<tr>
<td>3:15</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3:30</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>3:45</td>
<td>0</td>
<td>2</td>
</tr>
</tbody>
</table>

To classify Table 5; time column indicates when the counter entered the collected data. SB indicates the south direction in which the vehicles are going out of Kuwait City in the opposite direction of the studied section. NB represents the vehicles going to Kuwait City toward the intended section. Speed was classified into six categories starting from (0-40) km/hr and ending with speeds above (120) km/hr. The same method profile was made for Fahaheel road. Table 6 below, has the same direction classification SB and NB with the information gathered being the count of vehicles based on their lengths. This data was inputted in the model to ensure that the types of vehicles using the road were the same in the model vehicle classification based on length. This was used as an input to synchro to account for trucks, SUVs and buses.
Since Synchro is not designed to study traffic and safety behavior; input is a bit difficult. So, the driver behavior was inputted as the driver type in Synchro. Driver type in Synchro is a type of calibration to modify the model based on the parameters of each driver type. The main parameters that were changed included lane change, critical gap and gap acceptance. Each one of these parameters was adjusted based on the videos recording. Needless to say that adjusting these parameters to the actual statues produced some aggressive drivers in the model, which what is actually happening in real life and in the video. It was observed from the video that vehicles entering the main street from the side street or through the ramp were trying to force themselves to enter the main stream, which was considered an aggressive behavior. Drivers cannot be blamed for behaving in such a manner as there was no collector lane for them to enter in a more comfortable way. So, in this model the critical gap was reduced to simulate what is happening. This was assumed since the actual critical gap was nearly zero. For the gap acceptance factor, it is defined as the gap accepted by vehicles for permitted left turns and for right turns on red for a signalized intersection. By default, the program fixes these values from 1.15 to 0.85. It should be noted that the higher the value the more conservative the drivers are. In our case, the gap acceptance value was set to minimum since the flow on the street was
congested and the gap between vehicles was almost near to zero. Regarding the lane change, two lane changing parameters were added in Synchro. These were the average lane change time in seconds, which was basically used to account for maneuvers drivers do, and the lane change variance percentage.

4.4 Video Observation

The driver behavior was observed through video recording. It was noticed that due to the deficiency of acceleration lane, vehicles were trying to merge with the main road with at least one lane change in the peak hours when the movement was congested. Due to the congested flow on the street during the peak hours, the driver had to force his way in the main street. Even with the slow flow, it was noticed that drivers tried to change lanes at high speeds with a stop and go movement, which caused rear end and side type collisions between the vehicles trying to aggressively change the lane and the vehicles on the main road. This type of behavior was considered when the model was developed. Synchro is not meant to study the driver behavior in specific, so these behaviors were inputted in an indirect way. In Synchro there is a parameter called speed of turning which could be an indication of an aggressive driver. All aggressive turnings were counted, and they were evaluated as a percentage of the vehicles that were making a merge to the main road. Then based on the type of aggressive behavior, the value of the tuning speed was assumed.

For Synchro models, three models were done for both locations. The first one was the original case with the default driver parameters of the program, the second model included modifications to the driver parameters and the third one was the model with the suggested adjustment to the ramp. The suggestions were discussed with the ministry of interior consultant and contractor.

SSAM model was used to evaluate the traffic safety in the study area. Vehicles trajectories were extracted from the developed synchro model. SSAM model automatically identified the
location where traffic conflicts (i.e. potential accidents) could occur under a predefined time limit. In this paper the traffic conflicts were observed and recorded for each location three times: one for the model without the behavior parameters, the second was for the model after the driver’s parameters were inputted and the last time was for the model with the driver’s parameters and the new suggested modification to the existing location. All types of traffic conflict were considered (i.e. rear-end, lane change, among others).

5. Analysis of Results and Comparison

For Al Istiklal street, the focus was on the ramps area (diverge and merge). So, the model was developed for this section only. However, after running the simulation it was noticed that the flow of this traffic segment was not reflecting the actual traffic flow. So, a signalized intersection was included ahead of the desired section as shown in figure 3. The ramp with the study area is shown in figure 4.

![Figure 3: Model of Istiklal Road including the intersection](image)
The modified layout, only one modification was done to the original and current layout through adding a collector lane at the exit of the ramp, is shown in figure 5.

For Fahaheel Street, the location was along Fahaheel road at the ramp entering and exiting Mishrif area. Figures 6 and 7 show the overview of the model and the study area (ramps).
Regarding the modified layout in this location, the ramp was built with a collector lane. So, an alternative modification is needed. After discussing the possible solutions with the contractor engineers of the ministry of interior (MOI), the possibility of adding a second lane to the exit ramp was the most applicable solution. The modified model is shown in figures 8 and 9.
5.1 SSAM Results

SSAM was utilized to assess the safety degree of the models, identify the causes of the observed drivers’ behaviors as well as to evaluate the improvements done to the current condition. First, Istiklal Street was assessed using the default driver parameters then the model with the added collector lane was evaluated. The conflict points were recorded after which the models with
the added behavior modification were assessed and their conflict points were recorded and compared with the previous ones. The same process was repeated for the second location, Fahaheel road. However, in this model the modification was different as the study location already had a collector lane. So, the ramp exit (i.e. the entrance to the main road) was modified to two lanes. For Al Istiklal Street in the original layout without the behavior modification, running the model with the default settings in SSAM resulted in a total of 442 conflict points; 19 of them were crossing, 244 rear-end collisions and 179 lane change collisions. In the modified layout without the behavior modification, running the modified model in SSAM had resulted in 913 conflict points; 25 of them were crossing, 429 rear-end collisions and 459 lane change collisions. It was expected that the lane change collisions would rise since the vehicles in the main road were trying to change the lane before the entrance ramp as they were avoiding reducing the speed in the merging area. In the original layout with the behavior modification and after adding the behavior parameters to the same model, SSAM results had changed to 473 conflict points; 18 of them were crossing, 213 were rear-end and 242 lane change collisions. It should be mentioned that the only parameter that was changed was the driver’s parameter to eliminate any other effects. Finally, in the modified layout with the behavior modification, the same changes were done to the modified model and the results showed an increase in the conflict points as in the original model. The conflict points were 954 with 46 of them being crossing, 438 rear-end and 470 lane change collisions. The same pattern showed itself in this model with an increase in the conflict points.

The same process was done to the other location at Fahaheel Street for the original layout without the behavior modification. The results were a total of 319 conflicts, 275 of them were lane change, 36 rear end collisions and 8 crossing collisions. The results make sense since the video recording and site observations indicated a high break at the entrance of the ramp and at the exit. Most of the lane changes were in the exit. In the modified layout without the behavior
modification, the results were 574 collisions; 285 lane change, 274 rear ends and 15 crossing. It was noticed that this modification induced more conflict points and by default will produce more accidents. In the original layout with the behavior modification and after the behavior parameters were introduced, the total number of accidents increased to 375 total accidents. The main observation was that the lane change accidents decreased to 269 and the rear end increased to 100 accidents, which will give the designer a better understanding of the possible actions and the behavior of the drivers. As for the crossing accidents, it was decreased to 6 accidents. In the modified layout with the behavior modification, the modified scenario had resulted in a total of 1051 accidents; a dramatic increase in the total accidents with 329 lane change accidents, 716 rear-end and 6 crossing accidents. It was noticed that for both models, the modified and unmodified one, the rear-end collisions increased dramatically.

5.2 Results Summary

The table 7 below, shows the final summary for the two studied roads each with the modification to the layout and the behavior modifications done in the SSAM.

Table 7: Summary of the SSAM results

<table>
<thead>
<tr>
<th>Road name and status</th>
<th>Type of modification</th>
<th>Lane change conflict points</th>
<th>Rear-end accidents</th>
<th>Crossing</th>
<th>Total conflict points</th>
</tr>
</thead>
<tbody>
<tr>
<td>Istiklal Street</td>
<td>NON</td>
<td>19</td>
<td>224</td>
<td>179</td>
<td>442</td>
</tr>
<tr>
<td></td>
<td>Layout</td>
<td>25</td>
<td>429</td>
<td>459</td>
<td>913</td>
</tr>
<tr>
<td></td>
<td>Behavior</td>
<td>242</td>
<td>213</td>
<td>18</td>
<td>473</td>
</tr>
<tr>
<td></td>
<td>Both</td>
<td>470</td>
<td>438</td>
<td>46</td>
<td>954</td>
</tr>
<tr>
<td>Fahaheel Street</td>
<td>NON</td>
<td>275</td>
<td>36</td>
<td>8</td>
<td>319</td>
</tr>
<tr>
<td></td>
<td>Layout</td>
<td>285</td>
<td>274</td>
<td>15</td>
<td>574</td>
</tr>
<tr>
<td></td>
<td>Behavior</td>
<td>269</td>
<td>100</td>
<td>6</td>
<td>375</td>
</tr>
<tr>
<td></td>
<td>Both</td>
<td>329</td>
<td>716</td>
<td>6</td>
<td>1051</td>
</tr>
</tbody>
</table>

As shown in the previous table, the summary of the SSAM results showed that the effect of the driver’s behavior is clear. In Istiklal street, a significant increase of 7% in the total conflict points related to driver’s behavior factors was noticed. However, in Fahaheel street the driver’s
behavior had almost an effect of 17.55%. For the environmental effects and after running the simulation, Tables 8 & 9 show the amount of CO emissions for each location with different scenarios.

Table 8: Istiklal Road CO emissions

<table>
<thead>
<tr>
<th>Location Description</th>
<th>EBT CO Emissions (Kg)</th>
<th>WBR CO Emissions (Kg)</th>
<th>TOTAL CO Emissions (Kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Istiklal Road Without mod.</td>
<td>6.7</td>
<td>7.7</td>
<td>14.4</td>
</tr>
<tr>
<td>Istiklal Road With behavior mod.</td>
<td>6.74</td>
<td>7.1</td>
<td>13.8</td>
</tr>
<tr>
<td>Istiklal Road Without mod. 2nd lane</td>
<td>6.7</td>
<td>7.1</td>
<td>13.8</td>
</tr>
<tr>
<td>Istiklal Road With mod. 2nd lane</td>
<td>6.6</td>
<td>6.9</td>
<td>13.5</td>
</tr>
</tbody>
</table>

For the original layout, the total emissions were 14.4 Kg of CO. Then after applying the behavior adjustment; the total emissions were reduced to 13.8 Kg. So, the behavior alone influenced the environmental aspect even if it is small. For a larger scale model, it could have bigger and more visible results. Then, the adjustment to the same location was applied and the emissions were recorded again with 13.8 Kg. But in this case, the behavior adjustments were not included. After adding behavior adjustments; the emissions were reduced and reached 13.5 Kg.

Table 9: Fahaheel Road CO emissions

<table>
<thead>
<tr>
<th>Location Description</th>
<th>EBT CO Emissions (Kg)</th>
<th>WBR CO Emissions (Kg)</th>
<th>TOTAL CO Emissions (Kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fahaheel Road Without mod.</td>
<td>2.1</td>
<td>0.18</td>
<td>2.3</td>
</tr>
<tr>
<td>Fahaheel Road With behavior mod.</td>
<td>35</td>
<td>4.6</td>
<td>39.6</td>
</tr>
<tr>
<td>Fahaheel Road Without mod. 2nd lane</td>
<td>1.13</td>
<td>0.16</td>
<td>1.3</td>
</tr>
<tr>
<td>Fahaheel Road With mod. 2nd lane</td>
<td>36.5</td>
<td>4.6</td>
<td>41.1</td>
</tr>
</tbody>
</table>
The same analysis was done for Fahaheel road, but the variation in applying the behavior adjustments was large. So, multiple modifications were applied according to the video recording and the simulation was launched several times, but the results stayed the same. The huge increase of the CO emissions by the behavior alone is considered unreasonable in this location.

6. Conclusion

Traffic congestion in Kuwait causes considerable delay for all road users, which negatively impact their driving behavior, country economy, and quality of life. Congestion is the condition where the traffic demands exceeds the available roads capacity or the condition where the delays exceed a specific threshold. Weaving maneuvers at high traffic demand peaks over a short lane change distance forms a bottleneck on freeway and cause congestion. This paper studied the effect of the driver’s behavior on some of the suggested solutions for congested highways and especially at the exit and entering ramps. The evaluation was done through a computer software SSAM, and the solutions were discussed with the engineer responsible for the maintenance and construction of the road. In some cases, the increase in the possible accidents was significant and in some places was small. In any case, the effect was present, and this could lead to an inaccurate design or solution. It should be noted that both solutions increased the number of conflict points in both locations, and also increased the possible weaving rear-end side accidents. So, studying the highway condition may be more accurate if the behavior factor is considered. In this case, a 7% increase in the conflict points of Estiqlaal Street and a 17.55% increase in Fahaheel road were witnessed due to the sole effect of driver behavior without taking the change in layout into consideration.

7. References


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Prevention of Turning Accidents with Bicycle at Intersections

<table>
<thead>
<tr>
<th>AUTHOR</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
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<tr>
<td>RICHTER, Thomas</td>
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<td>Germany</td>
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</table>

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<th>POSITION</th>
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<tr>
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</table>

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KEYWORDS:
Accidents, Behavior, Cyclists, Right-turning, Cycle lane in middle position

ABSTRACT:
Turning accidents with bicycle are common and often very serious. In three research projects it was analyzed in what way the infrastructure and behavior from both road users influence the turning accident occurrence and how to reduce the conflict rate. The first project, initiated by the German Insurers Accident Research (UDV) was finalized in 2012 including right and left turning vehicles and cyclists driving straight ahead. The target of that research project was to give recommendations to improve safety of cycling infrastructure at intersections in built-up areas. The second project for the Federal Highway Research Institute (BASt) was finalized in 2014 and focused on the conflict between right-turning trucks and cyclists driving straight ahead. Both projects conclude recommendations concerning measures for infrastructural changes, driver assistance systems and behavior change to improve the traffic safety for cyclists.

Key recommendations are to improve the visibility between vehicles and cyclists for example by guiding the bicycle lane close to the carriageway and therefore improve the awareness of road users. Electronic assistant systems could support the driver to recognize cyclists. Nevertheless, due to the behavior of cyclists and obstacles in the side areas it continues to be challenging to detect them reliably. At complicated and complex intersections with high traffic volume of vehicles or cyclists it is recommendable to use different phases in signalization. In addition to that, infrastructure solutions like locating the straight-on bicycle lane on the left side of the right-turning lane could minimize the conflicts. This was the topic of the third research project, founded by the German Ministry of Transport and finished in 2017, which evaluated the effects.
Prevention of turning accidents with bicycle at intersections

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1 INTRODUCTION

In 2016, 26.4% of all accidents with personal damage in Germany are accidents with cyclists involved as reported in Statistisches Bundesamt (2018). Most of these accidents led to severe injuries or fatalities. Turning accidents have a share of around 20% of all cyclist-involved accidents and are the second leading accident cause after turn into or crossing accidents. These results are covered by several studies conducted to similar topics, e.g. Alrutz et al. (2009), Angenendt et al. (2005), Schnüll et al. (1992).

Law regulates it that turning cars have to give way to straight on driving cyclists. Nevertheless, the most frequently accident cause is “making mistakes while turning”. An accident analysis and a behaviour analysis should detect reasons for such mistakes. Related deficiencies could be on infrastructure-side, on on-board-side or on behaviour-side.

These deficiencies and accident causes were analysed by two separate research projects. The German Insurers Accident Research (UDV) as part of the German Insurance Association (GDV) have financed on project on conflicts between left and right turning vehicles and cyclists driving straight on. The other project, financed by the Federal Highway Research Institute (BASt), focused mainly on the special conflict between right turning trucks and cyclists driving straight ahead. The German Ministry of Transport has founded a third research project about one in the first two research projects recommended solution, a cycle lane in middle position.

2 METHODOLOGY

All projects started with an overview of the current state of science. Subsequently suitable German cities were researched and selected. Therefore provided accident data and local observations were used to define infrastructure attributes.

The German cities Darmstadt, Erfurt, Magdeburg and Muenster were selected for the first project on conflicts between left and right turning vehicles and cyclists driving straight on. Accident data between the years 2007 and 2009 were used for further analysis. Personal and telephone interviews and a behaviour observation were part of the project after an accidents analysis. For the project on conflicts between right turning trucks and cyclists driving straight ahead the German cities Berlin, Darmstadt, Magdeburg und Muenster were selected and data ranging from the years 2006 to 2011 used for further research. After an accident analysis driving tests in a truck simulator were conducted to evaluate the gaze movement before and during the turning process. In addition to that driving tests with and without a turning assistant were included. The results should identify infrastructural, board and driver-side measures and recommendations to reduce these turning-conflicts. For the third research project on cycle lanes in middle position test sides in Berlin, Hanover and Leipzig.

To compare the different infrastructure characteristics and the circumstances of accidents a classification of intersection types is essential. These classifications are selected according to infrastructure features like availability of traffic lights, on- or off-road bicycle lanes, separation distance of bicycle lanes and length and width of the different intersection elements.

3 LITERATURE REVIEW

According to the recommendations for bicycle infrastructure or the guidelines of signalization, few measures to prevent these conflicts were stated so far. Possible measures include that for example the bicycle stop line is marked in front of the car stop line (FGSV 2010, a) and that cyclists receive green light before other vehicles (FGSV 2010, b). These measures lead to a better detection of waiting cyclists at intersections even though it reduces only those conflicts,
which occur at the beginning of a green phase or when traffic participants need to stop because of a red light. No benefits are shown during green phases when vehicles approach the intersection with continuous drive.

Results from Schnuell et al. (1992) indicate that at intersections bicycle lanes routed directly next to the vehicle lane provide a higher safety level than those, which are further separated. With increasing distance between the vehicle and the bicycle lane the accident severity increases too. It is important to note that the opposite results apply for left turning accidents. The severity is increasing with an increasing distance between the vehicle and the bicycle lane. In general, the recommendations of bicycle infrastructure (FGSV 2010, a) suggest a distance between the cycle and the vehicle lane at intersections not more than 0.5 meters.

There are no specific recommendations for the special constellation of conflicts with right turning trucks and straight on driving cyclists the extent of cycle crossing. Different evasion movements conducted by the cyclist lead to contrary safety measures as described by Niewöhner and Berg (2004). In case the cyclist approaches the truck within a sharp angle the probability is higher that if a collision occurs the cyclist is falling down away from the truck. Additional mirrors can help preventing such collisions. In opposite if the approaching angle is wider a collision might cause less severe damage but the probability that the cyclist is falling down towards the truck increases. In such cases a direct view through the vehicle windows provides a higher chance to detect cyclists than additionally mounted mirrors. The size of the direct field of vision depends on several factors such as vehicle characteristics, tallness of the truck driver and tallness of the cyclist. An advantage by a minor extent of cycle crossing is a taper angle of collision.

The European Union has established various mirror systems for trucks. Due to these results, trucks have to implement in addition to the main outside mirrors a wide-angle-mirror, a near-area-mirror and a front mirror to minimize the blind spot area.

Niewöhner and Berg (2004) extracted two main scenarios for the accident collective of right turning trucks, straight driving cyclists and pedestrians and truck drivers stopping traffic-related before the turn. As soon as the traffic continuous to run, the truck moves and starts the right turn while a cyclist is approaching undetected due to the blind spot. In the other scenario the truck driver and the cyclist both need to stop at a red traffic light. After switching to the green light both continue their ride.

4 MACROSCOPIC ACCIDENT ANALYSIS

Due to various data bases and observation periods the accident analysis for each project was conducted separately, according to the bending process and to the involved people. Starting with the general evaluation of turning maneuvers and followed by the specific analysis of the right turning trucks.

The first research project covers the four German cities Darmstadt, Erfurt, Magdeburg and Muenster with data from 2007 to 2009. In total 79,756 accidents are listed where only a small proportion of 1.1% (873) are accidents with turning vehicles. Nearly 80% (693) of these turning accidents led to personal damages (fatalities, seriously and slightly injury). Automotive drivers have caused 91% of these accidents. In detail up to 94.5% are caused by “mistakes during turning”. For those cases where cyclists are partly to blame around 52% used the wrong way, 20% are caused by “another mistake” and 11% are caused ignoring traffic lights.

Male cyclists are more often included in an accident (60%) than female cyclists (40%). Concerning their driving performance there is no increased risk for male or female cyclists. The gender analysis of the vehicle drivers provides nearly the same results. 61% of the accidents involved male vehicle drivers and 39% females. Regarding their traffic volumes, the numbers for each gender compensate each other as female drivers share 35% driving kilometers causing 39% of the accidents. When it comes to analyzing the results by age cyclists between 21 and 34 years provide a two times higher risk than other age groups. Vehicle drivers have a two times higher risk of being involved in a turning accident between the age of 18-20 and 65-75+ year.

The analysis of the temporal distribution outlines hydrographs of days, weeks and years. Based on these most accidents happen during the cycle season from March to October. Furthermore, most accidents happen during the week. At weekends the amount of accidents decrease due to lower traffic volumes. In addition to that, most accidents happen between 6 am and 8 pm with the highest amount of accidents during the morning and afternoon rush hour. During the night and early morning hours fewer accidents occur. Variations between the different cities are nearly non-existing. Also, the analysis concerning the light and weather conditions are negligible. Basically most accidents happen during daylight (81%) and on dry roadways (80%). Only few accidents happen during twilight and darkness or on wet and icy roadways.

The macroscopic accident analysis can be summed up as the following:
- **Raised risk**
  - Right turn (2/3) vs. left turn (1/3) vehicle
  - Intersections with traffic lights (nearly 70%)
  - 21–34 years old cyclists nearly double (concerning to their cycle miles)
  - 18–20 and 65+ years old vehicle drivers (concerning to their driving miles)
  - Cyclists driving on the pavement or/and on the wrong direction
- **Main cause**
  - 90% cars and 10% trucks
  - Out of truck 80% vans and delivering vehicles
- **Obstructive view at 41% of intersections**
  - Increasing obstructive view with increasing distance to the vehicle lane:
    - 2 - 4 meters in between: 69%
    - > 4 meters in between: 61%

**5 INFLUENCE OF INFRASTRUCTURE**

In addition to the analysis of the accident data on-site inspections of 150 intersections were carried out to find more clues regarding accident causes. Therefore, characteristics like which kind of cycle facility, infrastructural guidance of the bicycle lane next to the road, condition and execution of the crossing, presence of traffic lights or occurrence of an obstructive view have been analyzed.

Generally said, one third are left turning accidents and two thirds are accidents with right turning vehicles. Furthermore, nearly three out of four accidents happened at intersections with traffic lights (n=150). Most intersections have a two-phases signalization, which means that cyclists and vehicle drivers receive green light at same time.

Slopes were detected in 15% of the analyzed crossings and 6% provide a large radius. Both characteristics can lead to higher vehicle speed. Another 6% of the intersections were perceived as large and inconclusive. A further 3% of the junctions have shown wornout markings for the cycle crossing. In addition to the analyzed intersections with accidents, junctions without accidents have been reviewed. At these 17 equivalents none of those characteristics has been found.

At 41% of the intersections sight disabilities were identified. This obstructive view could be provoked because of parking vehicles, trees, bus stations etc. The obstructive view is higher with an increasing distance from the bicycle lane to the vehicle lane. When the bicycle lane is two to four meters besides the vehicle lane, 69% of intersections underlie obstructive vision. If the bicycle lane is more than four meters away, a sight disability of 61% is endogenous.

The proportion of accidents with right turning trucks covers less than 10% when the turning maneuvers between vehicles and bicycles are basic population. That means that nearly every 10th turning accidents between vehicles and cyclists happened with truck involvement and needs a special investigation.

**6 ACCIDENTS WITH RIGHT TURNING TRUCKS**

Accidents with right turning trucks and cyclists straight on driving do not interfere often (755 cases) but if conflicts happen they lead to severe damages (nearly 80% with personal injury). Fatalities make about 2% (16 cases) of those accidents, nearly 10% (74 cases) cause severe injuries and over 67% (508 cases) involve slightly injured persons. The remaining 20% are accidents with material damage.

Most accidents (92%) happen with vans / delivery trucks / trucks without trailers. Just 8% of the turning accidents with trucks are accidents with heavy semitrailer trucks. Concerning the severity of such accidents, those with semitrailer trucks result in more serious damages than those with vans or delivery trucks. In 19% of the accidents with severely injured persons and 37.5% of the accident with fatalities a semitrailer truck was involved.

The main cause of 96% of these accidents are trucks where the most common reason was an incident turning maneuver. Cyclists make the main cause for only 3% of the accidents. Thereby the wrong usage of roadways is the most frequent reason to provoke an accident. Another 4.6% are caused because of driving in the opposite direction. These data were deleted subsequently because driving in the opposite direction does not provoke typical blind spot problems relating to right turning trucks and cyclists driving straight ahead. Only 1% of accidents happened because cyclists violated a red traffic light.
The main problem seems to be that none of the drivers previously came from a standing position. That means cyclists and vehicle drivers pass through the intersection without the need to stop. This could lead to scenarios were the cyclist is located in the blind spot of the truck and both vehicles approach the intersection with almost the same speed. The scenario were both drivers have to stop traffic-related before they continue on is not a problem for this accident collective. Therefor the large amount of mirrors and the direct view through the windows seems to be very useful.

Male cyclists are more often included in accidents (61%) in Berlin than female cyclists (39%). Concerning their driving performance there is no increased risk for male or female cyclists. Male cyclists in the cities Darmstadt, Magdeburg and Muenster are less often included in an accident (48%) than female cyclists (52%). Concerning driving kilometers female cyclists even have an increased risk to be involved in a conflict (42% of the driving kilometers) than male cyclists (58%). The gender analysis of the truck drivers contains nearly the same results. Male truck drivers make 91% of the accidents while female drivers cover only 3% (another 6% are unable to attribute because of hit and run accidents). Based on the institute of employment market and occupational research, the division between male and female truck drivers in 2010 was 96% to 4%. There exists no additional differentiation regarding their driving kilometers. The age-related analysis results an increased risk for cyclists between 21 and 34 years and in Berlin ancillary the 35-44 years old cyclists. Vehicle drivers have a raised risk according to their driving kilometers in the age of 18-24 (nearly double the risk of other age groups) and 25-34 (around one third).

7 BEHAVIOR AND CONFLICTS

Altogether 43 intersections were selected to be monitored via cameras. The selection procedure was based on the evaluated infrastructure characteristics. Every intersection was surveyed for 3 hours, either during the morning rush hour from 7 to 10 am or during the afternoon peak from 2 to 5 pm. Afterwards the videos were evaluated manually by estimating every single situation. Thereby interactions and conflicts were analyzed at intersections with and without (equivalent) accidents. Comparing this two attributes, at equivalent intersections a conflict involvement rate of 6.5% has been obtained (Table 1). At accident-prone junctions a higher conflict involvement rate of 10.7% has been determined. (Kolrep-Rometsch et al. 2013)

Table 1: Comparison of intersections with (accident-prone) and without (equivalent) accidents, Source: Kolrep-Rometsch et al. (2013)

<table>
<thead>
<tr>
<th>observed intersections</th>
<th>equivalent</th>
<th>accident-prone</th>
</tr>
</thead>
<tbody>
<tr>
<td>amount of interaction</td>
<td>17</td>
<td>26</td>
</tr>
<tr>
<td>total amount of conflicts</td>
<td>118</td>
<td>590</td>
</tr>
<tr>
<td>average</td>
<td>0.5</td>
<td>2.4</td>
</tr>
<tr>
<td>standard deviation</td>
<td>0.8</td>
<td>2.5</td>
</tr>
<tr>
<td>conflict involvement rate</td>
<td>6.8%</td>
<td>10.7%</td>
</tr>
</tbody>
</table>

Furthermore, several traffic behaviors such as indicate rate, speed reduction and shoulder check were captured for every occurrence independent from conflict or non-conflict situations. The indicate rate was in both cases high (98% in general and 96% in a conflict). Reducing the speed while turning happened more often overall (91%) than in a conflict (70%). The look over the right shoulder before turning right occurs less often in a conflict (67%) than in general (82%). (Kolrep-Rometsch et al. 2013)

The evaluation of the signalization phases provides more interesting results which can be seen in table 2. The most common situation is that both drivers start driving again after stopping at a red traffic light. However, the conflict rate case is the lowest (3.2%) for this case. On the contrary, the conflict rate is the highest (39.8%) if the vehicle driver starts driving after a red traffic light and the bicycle rider passes through without stopping. In addition to that, the conflict involvement rate is six times higher if the cars are turning in a convoy (38.8%) than a single vehicle turns right (6.1%). (Kolrep-Rometsch et al. 2013)

Table 2: Conflict involvement rate concerning the signalisation phase, Source: Kolrep-Rometsch et al. (2013)
Concerning traffic regulations there exist different perceptions. On one hand most drivers know that cyclists driving straight on have priority and right turning vehicles have to give way (97.3%). On the other hand, the knowledge about the obligatory using of bicycle lanes is insufficient. Just 14% of the vehicle drivers and bicycle riders know that a bicycle lane is only mandatory to use once a traffic sign is mounted. If there is no road sign cyclists can choose between driving at the vehicle lane or on the separate bicycle lane. (Kolrep-Rometsch et al. 2013)

### 8 BEHAVIOR OF TRUCKS AND USE OF TURN-OFF ASSISTANT

As part of the driving simulator study routes have been implemented which are different regarding their infrastructure design. The driving and gaze behavior during the right turning of 48 study participants were analyzed. In addition to that, the influence of a turn-off assistant was tested, which informed half of the drivers in turning situations by an audible and visual warning when a cyclist was approaching. The other half of driving tests happened without any assistant system. Comparing the turning situations with and without simulated turn-off assistant, hardly any differences could be found in the driving parameters or gaze data.

Because of the coding and the different way of driving in some cases an interaction between right turning trucks and straight on driving cyclists could not be ensured. For example, if the speed of the truck driver is more or less than planned, the interaction did not happen because either the cyclist crossed already the intersection or the cyclist was located behind the truck driver. Another Problem was the minor number of evaluable trips because of simulator illness. (Richter et al. 2015)

The lowest speed was detected when cyclists do not have their own bicycle lane. Furthermore, the speed is lower if the distance between the bicycle and the vehicle lane is decreasing. Even the relative gaze frequency to the cyclist through the right window is increasing when the distance between the lanes is decreasing. Concerning the gaze behavior into the mirrors there were no abnormalities. None cycling infrastructure is suspicious to that token. (Richter et al. 2015)

### 9 RECOMMENDATIONS AND MEASURES

Based on the accident and drivers behavior analysis following recommendations for infrastructure design can be given which are partly already included in German guidelines. A rather small distance between the bicycle and the vehicle lane at intersections should be preferred because of the increasing obstructive view with increasing distance between vehicle lanes which is verified by both the microscopic analyses and the behavior analyses. In addition to that, intersections should be clearly, quickly recognizable and comprehensibly designed. The marking of bicycle crossings should be obvious and if required additionally marked red (e.g. high accident rate or high traffic intensity). Obstructions (e.g. parking vehicles, trees, advertising panels etc.) should be avoided. In general regulations which can lead to conflicts among traffic participants should be avoided. For example, vehicle drivers should not need to expect cyclist driving on the pavement and therefore additional traffic signs which grant cyclists the usage of the pavement should be avoided. A signalization providing separate phases for cyclists driving straight and right turning streams is recommended if the traffic volume of vehicles or cyclists is high or an obstructive view exists. Furthermore, when the speed of vehicles and cyclists is high because of a downhill road or a big radius, a separated signalization is suggested. This is a promising way to reduce accidents, even those who happened because of the continual drive. A signalization with separated phases is required if there exists more than one turning lane for vehicles (FGSV 2010, b). Therefore, it is necessary to implement the standards which are given in the guidelines. In most cases junctions display some compromises which can be responsible for accidents or conflicts.
It can be captured that the truck drivers already several mirroring systems which should be on one hand on the right position and on the other hand the gaze to the mirrors which helps in the right moment to detect a cyclist. Nevertheless, it needs to be considered that every additional mirror enlarges the field of indirect view but reduces the field of direct view. Furthermore, every further mirror claims an extra attention for the truck drivers. Therefore, a mandatory implementation of additional mirror systems on trucks should be avoided.

The fixed mirrors at intersections are only suitable for the special situation when the truck and the bicycle rider have to stop at a red traffic light. However, this represents a minor problem at the accident collective of right turning trucks and cyclist driving straight. Although the direct costs for a mirror are rather small, within the areal distribution of those accidents the cost would be very high. Nevertheless, at black spot intersections that measure could certainly contribute to reduce the number of accidents.

Moreover, an ancillary measure is the sensitization of all traffic participants because of the analyzed data that highlights the increasing risk of involving in an accident if the shoulder check is missing. In many cases the bicycle riders do not realize the danger and insist on their right of way, regardless of the problem that in some cases the car or truck driver cannot see the cyclists. Furthermore, a high attention should be paid to the danger of blind spots in driver trainings, especially for truck drivers. In addition to that, an enhanced awareness for cyclists should be trained even if they have the right of way. Further explanations for the obligatory usage of the bicycle lanes have to be suggested because of the questionnaire results. A considerate handling in traffic may affect higher safety.

As general result of all the parts of the study is can be said that risks increase when cyclists and vehicle drivers do not need to make traffic-related stops (red traffic light). Most accidents happen during continuous drive (without stopping) and also the conflict rate is much higher (Table 2) compared to situations including stops at the red traffic light. If the cyclists are covered in the blind spot area while driving, the truck driver has limited chances to detect the cyclist while turning which leads to dangerous situations on both sides. The measures included in the German guidelines are more or less for conflicts including intersection stops. In the future there is a strong need for measures preventing turning accidents for green light phases when right turning vehicles and cyclists going straight pass through the intersection without the need to stop traffic-related.

One recommended measure was a cycle lane in center position between the right-turning lane and the straight on lane for the vehicles. The idea was, that all problems and conflicts with turning vehicles and with the sight can be banned and the additional conflicts in the linking area are not so problematic.

![Figure 1: cycle lane in center position / through bike lane](image)

The results of our third study point out, that a cycle lane in center position cannot improve the traffic safety in general and such lanes only should be used in special situations and with a special length and width. A lot of accidents are shift to the linking area, where the share of serious accidents is higher than at the turning point. Also there are more conflicts investigated at the linking area. The subjective cognition of cycle lanes in center position by the cyclist especially when they are short or narrow is also very negative. Due to this cognition a lot of cyclist do not use the cycle lane but the footpath, where they cause new conflicts.

Table 3: before and after study concerning cycle lanes in center position / trough bike lanes
Cycle lanes in center position have only in special situations a positive effect on traffic safety. Just if the number of cyclist is high and the number of turning vehicle is low to medium they should be used. In case of using they should have a medium length of 40 to 60 m a width of 2.1 m and they should be colored marked.

Another recommended system to prevent right turning accidents is a blind spot detection system for trucks. There are several systems in the market and at the moment in the European Union there is a discussion about the mandatory introduction of the system. The problems are, that there cannot be a 100% detection and prevention due to several reasons. On one hand side the detection rate cannot be 100% because cyclist can drive behind parked cars and delivering vehicles. On the other hand cyclist and also pedestrians can move and turn so suddenly, that a collision cannot be prevented. But if the system can prevent more than 90% of all serious accidents they should be obligatory and for the remaining accidents there must be a legal framework, without a 100% product liability.

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Protection of Vulnerable Road users on National Routes in South Africa

Towards Zero deaths (Protecting Vulnerable Road Users)

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SANRAL  
Decade of Action  
Pedestrians

Urbanization is an unavoidable worldwide phenomenon. Urban developments termed “Urban Sprawl” along major routes seem to be a trend which has been adopted in South Africa. Developments are beneficial to a country’s economy and aids in job creation which indirectly creates pedestrian movements.

Pedestrian fatalities are a major crisis in countries globally. Road safety is of paramount importance to South Africa. The South African National Roads Agency (SANRAL) has made it one its strategic objectives to focus on improving Road Safety.

This paper will discuss the strategy adopted to undertake the improvements implemented for protection of vulnerable road users along a portion of the National Route N2 in South Africa. Design proposals will be compared and discussed which will aid in illustrating why the most favourable option was selected. This paper will also highlight how SANRAL attempts to harmonise the existence of various road users within its national road system.
INTRODUCTION

In 2015, South Africa was regarded to be among the highest road fatality countries in the world. Due to the increasingly high number of road deaths, South Africa was ranked 6th internationally. This is shown below in Figure 1.

![Figure 1](image)

Figure 1. Number of fatalities per country across the globe due to road accidents in 2015
(World Atlas, December 2015)

However, in 2017 South Africa did not feature among the top 30 countries with the highest road fatalities internationally. This is shown below in Figure 2. Two reasons for this are that South Africa has had interventions to reduce the fatalities or/and other countries have had increasing fatalities.

![Figure 2](image)

Figure 2. Number of fatalities per country across the globe due to road accidents in 2017

Figure 2 above shows that road fatalities is a global problem and mainly affecting developing countries. To remedy this problem, the General Assembly Resolution 64/255 adopted on 2 March 2010 requested that the World Health Organization (WHO) and the United Nations (UN) regional commissions, in cooperation with the United Nations Road Safety Collaboration and various stake holders, prepare a Plan of Action as a guideline document to support and assist the drive to reduce road fatalities termed the “Decade of Action”.

The Decade Plan of Action was officially launched in May 2011 to assist and attempt to reduce the number of road fatalities by 50% globally by year 2020.

Based on the recommendations of the World Health Organisation’s world report on road traffic injury prevention, countries were encouraged to focus on the following five road safety pillars, namely (Implementing the 2011-2020 Decade of Action in Sub-Saharan Africa, 2010):

- Pillar 1: Road Safety Management
- Pillar 2: Safer Roads and Mobility
- Pillar 3: Safer Vehicles
- Pillar 4: Safer Road Users
- Pillar 5: Post-Crash Response
South Africa had developed a Road Safety Manual \textit{(SARSM, 1999)} to assist road authorities with the evaluation of traffic operations and assessment of road safety aspects along the road network nationally.

**BACKGROUND**

The SARSM was published in 1999 as a draft document and consists of the following volumes:

- Volume 1: Principles and Policies
- Volume 2: Road Safety Engineering Assessment on Rural Roads
- Volume 3: Road Safety Engineering Assessment on Urban Roads
- Volume 4: Road Safety Audits
- Volume 5: Remedial Measures and Evaluation
- Volume 6: Roadside Hazard management
- Volume 7: Design for Safety

In 1998 the South African National Road Agency SOC Ltd (SANRAL) was established as an independent, statutory company with a distinct mandate to finance, improve, manage and maintain the national road network within South Africa. SANRAL currently manages 22,197 km (2017) of national roads in South Africa. SANRAL’s direct sphere of influence relates specifically to the Pillars of Safer Roads and Mobility, Safer Road Users, and Post-Crash Response. For Safer Roads and Mobility reference to road infrastructure interventions relates directly on the road network, for Safer Road Users reference is made to Road Safety Education and Awareness Programmes, and for Post-Crash Responses reference is made to the rollout and operations of Road Incident Management Systems on the entire national road network. These are the key focus areas where SANRAL will continue to promote and improve road safety for all road users.

Even though South Africa is no longer among the top 10 highest road fatality countries internationally as shown in Figure 2 above, this should not necessarily be interpreted that the number of fatalities in South Africa have reduced by interventions adopted, this could potentially be because the number of road fatalities in other countries have increased. The fact of the matter is that there is still a very large number of road fatalities still occurring in South Africa. More importantly the number of pedestrian fatalities is still an unacceptably high percentage of total fatalities which is of serious concern. This is shown below in Figure 3. A key focus on pedestrian safety is of paramount importance and will be discussed in detail in the contents of this paper.

![Figure 3. Number of fatalities in South Africa 2008 to 2017 (RTMC Calendar Report, 2018)](image)

Figure 3 above, shows that the number of road fatalities have increased since 2015, from 12944 to 14050 fatalities in 2017 i.e. an 8.5% increase. Hence, emphasizing the fact that road fatalities is still of serious concern. According to the Figure 3 14050 road fatalities occurred, which is marginally lower than 2016 but still higher than any year between 2008 and 2015. From the 14050 road fatalities in 2017, 38% (5339) of these fatalities were pedestrians.

Over the last 10 years i.e. from 2008 to 2017 approximately 137 000 road fatalities occurred in South Africa, of which 36% (49636) were pedestrians. It is evident that approximately one third of fatalities occurring
from road accidents are pedestrian related who are the vulnerable road users. Every fatality has a direct impact to the economy of the country. Table 1 below shows the average cost of a fatality in South Africa from different authorities.

Table 1. Average Fatality Cost in South Africa

<table>
<thead>
<tr>
<th>Average Fatality Cost in South Africa (2017)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R 1 160 654</td>
</tr>
</tbody>
</table>

The average cost of a fatality is related to the total value that a person would have contributed to the economy of South Africa over his/her life time. Based on the average cost of approximately R 1 500 000 per fatality in South Africa, the total loss to the economy in 2017 alone was approximately R 21.0 billion due to road fatalities (14050 fatalities). Between 2008 and 2017 a total loss to the economy was approximately R 205 billion due to road fatalities (137 000 fatalities).

Analysing the number of pedestrian fatalities independently, in 2017 a loss of R 8 billion was incurred, and over the last decade South Africa’s economy suffered an approximate loss of R 75 billion. According to the Pietermaritzburg Agency for Community Social Action (PACSA), a cost to feed a family (consisting of 2 adults, 2 children and 1 elder) is approximately R 38,400 per annum (R 3 144/month) (PACSA, April 2018). Hence, in 2017, and over the last decade a total of 200,000 and 1.9 million families respectively could have been sustained with the loss of income to the country due to pedestrian fatalities, assuming all fatalities were of the bread winner of the household. A high percentage is however kids.

Alternatively, this loss to the economy could have been utilized in various other sectors such as, infrastructure development, health services, housing, education etc. Hence the reduction in pedestrian fatalities will strengthen the economy of the country and allow an opportunity for South Africa to develop sectors that requires assistance.

A pedestrian can be defined as a person on foot rather than travelling by car. One would expect that this would be the safest way to travel, since you are traveling at low speed, no risk of any mechanical failure, like in automobiles, aircrafts etc. According to one of South Africa’s largest road safety campaigns “Arrive Alive” the contributing factors to pedestrian fatalities in South Africa are as follows (Arrive Alive, 2018):

- Drunk pedestrians – Pedestrians who are intoxicated and impaired with alcohol or/and drugs.
- Reckless/ lawless pedestrians – Those pedestrians taking chances by running across the roads and freeways underestimating the speed of vehicles. (Pedestrians are in fact not legally permitted to be on freeways.)
- Distracted pedestrians – Pedestrians failing to pay attention to traffic whilst distracted by cellular phones, music, etc.
- Pedestrians ignoring/disobeying traffic light signals – crossing the road against an orange or red light.
- The “Poorly visible” pedestrians: Those walking on the shoulder of the road not wearing high visibility or reflective clothing.
- Pedestrians walking on the road instead of on the verge or sidewalk, often on the left side of the road where they are travelling in the same direction as the traffic (back towards oncoming traffic) and therefore cannot see approaching traffic.
- Pedestrian inattentiveness - Where we find pedestrians such as children chase after something, e.g. a ball or hat which might be on the surface of the road.
- Lack of supervision / Younger pedestrians left unattended – This includes our younger vulnerable child pedestrians not supervised & left on their own at roadside.
- Crime forcing pedestrians onto the road surface – Those pedestrians robbed or attacked and trying to
get away from the attackers, ending up as victim of a road crash.

- Victims of reckless driving – those instances where drivers lose control and pedestrians, through no fault of their own, are hit by these vehicles.

From the above it is evident that only certain contributing factors can be addressed from an engineering perspective and majority of these factors can only addressed through education forums.

According to a survey released by the South African Institute of Race Relations, people living in urban areas increased from 52% in 1990 to 63% in 2011, simultaneously the people living in rural areas decreased by 48% and 38% over the same period. According to Property24 one of South Africa’s most reputable estate agent portals, rapid economic growths in metropolitan areas are one of the major causes of urbanization in South Africa. Shown below in Figure 4 is the South Africa’s Urbanization percentage in South Africa between 2006 and 2016.

![Figure 4. South Africa Urbanization from 2006 to 2016 (Statista 2018)](image)

Figure 4 above, shows that there has been a constant increase in urbanization over the last decade. People tend to migrate to cities to gain economic independence and freedom. This growth spurt in population leaves urban areas densely populated. Urbanization or “Urban Sprawl” also lead to low income settlements occupying urban environment. Generally, in South Africa, these settlements are found in close proximity to roads as this provides easy access to public transport due to a vast number of people not owning private vehicles. Most pedestrian related accidents are concentrated at these localities. While freeways are designed not to accommodate pedestrians, South Africa’s history is that communities are located adjacent to freeways. Therefore, the reality is that they must be accommodated to ensure their safety.

For the purpose of this paper, the strategy adopted to undertake the improvements implemented for protection of vulnerable road users along a portion of the National Route N2 between Ballito and Stanger in the KwaZulu Natal Province in South Africa will be discussed. Design proposals will be compared and discussed which will aid in illustrating why the most favourable option was selected. This paper will also highlight how SANRAL attempts to harmonise the existence of various road users within a road system. A locality map is shown below in Figure 5.
A traffic analysis of this section of the N2 has indicated the need for a capacity upgrade by the addition of lanes as well as the upgrade of Interchanges. However, along this section there have also been reports of a number of pedestrian fatalities taking place. Hence, a safety improvement project with the mandate of addressing pedestrian fatalities was identified and sanctioned. SANRAL appointed a Consulting Engineering Firm (CEF) to undertake investigations to determine the possible cause of the accidents and propose possible solutions.

METHODOLOGY

The approach and methodology adopted to obtain an optimum solution to the ongoing pedestrian accidents will be discussed under this section of the paper. It should be noted that even though SANRAL was the client and appointed a CEF to undertake the investigations and design, SANRAL played a fundamental and key role in the management of the CEF, interpreting the findings and decision making.

As part of the investigation SANRAL adopted the following methodology:

a) Pedestrian Accident Analysis
   - Pedestrian accident statistics analysis
   - Visual Inspection
   - Pedestrian counts
   - Vehicle travelling speeds
   - Common ‘red spots’ areas for pedestrian incidents

b) Potential causes of accidents that can be remedied from an engineering perspective

c) Design Development and Adoption of proposals at:
   - N2 Mainline
   - Interchanges
   - Lay-byes
   - Security wall at Toll Plaza

It should also be noted that all projects are governed by a financial budget, it is the responsibility of the CEF to produce a financially viable solution. Notwithstanding, this statement should not be interpreted that South Africa and SANRAL will compromise road safety as a result of financial constraints. All that is required is that all proposals and solutions are engineered in such a manner as to optimize and value engineer the best possible solution.

From the methodological approach as indicated above, it was envisaged that the outcome of such an approach would enable one to determine the locality of the accidents, potential causes as well as the desire lines of the pedestrians. This approach would allow SANRAL to identify and reduce the pedestrian accidents and reduce the number of conflict points and hopefully fatalities.

RESULTS AND ANALYSIS

a) Pedestrian Accident Analysis

As part of the pedestrian accident analysis the initial investigation was to obtain any accident records from national and local traffic authorities and analyse the existing accident records. Accident records were obtained from four (4) sources, namely:

- South African Police Services (SAPS)
- Road Traffic Inspectorate (RTI) – (Road Traffic Police)
- Road Traffic Management Corporation – (RTMC)
- Intertoll – (Operator of the Toll on this section of the N2)

Fortunately, SANRAL has a single database referred to as the Integrated Transport Information System (ITIS) where all accident data from the abovementioned sources are uploaded on a regular basis which is
shown in Figure 6 below. However, it should be noted that accidents still occur and may not be reported and documented.

Figure 6. Pedestrian Related Accidents ‘Red Spots’

Figure 6 above, shows between 2010 and 2018 there has been an excess of 40 documented pedestrian related accidents between Ballito and Groutville of which 18 were fatal.

From the statistics shown in Figure 4 above, it was then possible to identify possible locations to undertake pedestrian counts and determine the exact capacity of infrastructure that would be required to reduce the number of accidents and pedestrian fatalities.

It was fundamentally important that pedestrian counts were undertaken in order to understand the magnitude of pedestrian movements between two local towns of Ballito and Groutville as well as across all interchanges. This information also allowed us to identify the volume of pedestrians where no proper pedestrian facilities are in place. Pedestrian counts were undertaken at the following locations as shown in Table 2 below.

Table 2. Summary of Pedestrian Counts

<table>
<thead>
<tr>
<th>Counts at Freeway Level (Average over 2 days in 2017)</th>
<th>Pedestrians (No.)</th>
<th>Cyclists (No.)</th>
<th>TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Count Station</td>
<td>Location</td>
<td>North-Bound</td>
<td>South-Bound</td>
</tr>
<tr>
<td><strong>ALONG THE N2</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Between Ballito I/C and Shakas I/C</td>
<td>15</td>
<td>36</td>
</tr>
<tr>
<td>2</td>
<td>Between Shakas I/C and Salt Rock I/C</td>
<td>87</td>
<td>90</td>
</tr>
<tr>
<td>3</td>
<td>Between Salt Rock I/C and Tinley Manor I/C</td>
<td>10</td>
<td>14</td>
</tr>
<tr>
<td>4</td>
<td>Between Mvoti Toll Plaza and Groutville I/C</td>
<td>317</td>
<td>86</td>
</tr>
<tr>
<td><strong>ACROSS THE CROSS-ROADS OF THE INTERCHANGES</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Shakas I/C</td>
<td>1303</td>
<td>996</td>
</tr>
<tr>
<td>6</td>
<td>Salt Rock I/C</td>
<td>706</td>
<td>540</td>
</tr>
</tbody>
</table>
Table 2 above shows that there are high pedestrian movements along this section of the N2 corridor and interchanges and proper pedestrian facilities with adequate pedestrian protection measures from vehicles should be implemented to reduce pedestrian accidents and ultimately protect the vulnerable road user.

From the pedestrian counts provided in Table 2 above, the following locations were identified as hazardous areas and required an immediate intervention:

- Along the N2 between Ballito I/C and Salt Rock I/C
- Shakas Rock I/C
- Salt Rock I/C
- Along the N2 Between Mvoti Toll Plaza and Groutville Interchange

Figure 7 below shows a locality map that will assist the reader to identify to position of the hazardous locations discussed above.

![Locality Map](image)

**Figure 7. Locality Map**

b) **Potential causes of accidents that can be remedied from an engineering perspective**

A site inspection was undertaken to determine the possible causes of accidents and pedestrian fatalities that could potentially be solved by engineering solutions. The findings of the site inspection can be summarized as follows:

- There are no formal pedestrian walkway facilities along the N2 hence pedestrians tend to walk on the slow lane shoulder and are placed at a greater risk as they are walking alongside the heavy and high-speed vehicles,

- Pedestrian movements along the N2 carriageway between Ballito I/C and Salt Rock I/C are mainly as a result of urbanization. There has been rapid growth in developments along the N2 corridor especially been concentrated between these two Interchanges. People find it much easier and cheaper to commute by foot and
bicycles due to the short distance between these two interchanges.

- Pedestrians were found to walk across the N2 carriageway as it appeared to be the shortest route to their desired destination. This crossing movement is one of the major contributors to pedestrian fatalities between Ballito I/C and Salt Rock I/C as well as between Mvoti Toll and Groutville I/C.

- It was also identified that passenger transportation vehicles (taxis) were stopping along the slow shoulder of the N2 at the Salt Rock and Shakas Rock interchange to pick up and drop off passengers. This poses a safety hazard to both pedestrians as well as motorists using the N2. This was identified as one of the potential attractors for pedestrians crossing the N2.

- The problem at all interchanges is that the pedestrians have no option but to walk in the surfaced shoulder of the cross-road, with their only “protection” being a yellow line marking as there are no formal sidewalks.

- Between the Mvoti Toll Plaza and Groutville interchange, it was noticed that a residential development was in close proximity to the N2. At this location, it was identified that majority of the pedestrians were children without adult supervision. Children are of a serious concern as more often than not are complacent regarding safety.

Further investigations on the speed profiles along this section of the N2 were undertaken previously. The results of this investigation are shown in Table 3 below.

Table 3. Indicating Speed Profiles

<table>
<thead>
<tr>
<th>TEL STATION</th>
<th>SECTION</th>
<th>KM</th>
<th>AVERAGE SPEED (Km)</th>
<th>85% SPEED</th>
</tr>
</thead>
<tbody>
<tr>
<td>1279</td>
<td>Salt Rock Plz</td>
<td>9.6</td>
<td>107.5</td>
<td>121.9</td>
</tr>
<tr>
<td>799</td>
<td>Unfill I/C</td>
<td>11.7</td>
<td>107.5</td>
<td>121.9</td>
</tr>
<tr>
<td>2231</td>
<td>Mvoti Plaza 2</td>
<td>21.6</td>
<td>107.5</td>
<td>121.9</td>
</tr>
<tr>
<td>1069</td>
<td>Groutville I/C</td>
<td>33.4</td>
<td>107.5</td>
<td>121.9</td>
</tr>
<tr>
<td>801</td>
<td>Stanger I/C</td>
<td>30.0</td>
<td>107.5</td>
<td>121.9</td>
</tr>
<tr>
<td>802</td>
<td>Zinkwazi</td>
<td>41.6</td>
<td>107.5</td>
<td>121.9</td>
</tr>
</tbody>
</table>

From Table 3 above, it is evident that the vehicles travelling along this section of the N2 are travelling at high speeds which vary from 121km/h to 126km/h. This emphasis the need for proper pedestrian facilities which would reduce vehicular and pedestrian conflicts. With reference to Table 2, “Summary of Pedestrian Counts” above, an approximate total of 769 pedestrians are walking along the N2, which is of great concern.

In order to solve the problems identified above SANRAL advised the appointed service provider to submit proposals that would ensure pedestrians are able to walk in a safer environment which potentially would reduce the number of pedestrian accidents and fatalities. The proposed solutions of the above finding will be discussed under the design development phase below.

c) **Design Development phase**

- **N2 MAINLINE- CARRIAGEWAY BETWEEN BALLITO I/C AND SALT ROCK I/C**

Due to the high number of pedestrian movements between the Ballito I/C and Salt Rock I/C, it was proposed that a walkway be formalised in order to accommodate the pedestrian and cyclists.

A desire line analysis was undertaken in order to determine the routes where the pedestrians are walking. This enabled the CEF to determine which sides of the N2 to accommodate pedestrians with walkways. This exercise was fundamental, as optimization of designs are always required for value engineered solutions. Regarding the pedestrians illegally crossing the N2, a security fence was proposed to be installed along the median of the N2. Pedestrians could however cross at safe crossing points shown in the layout plan of the
walkways and the pedestrian fence shown in Figure 8 below.

Figure 8. Layout Plan of Walkways and pedestrian Fencing between Ballito I/C and Salt Rock Interchange

WALKWAYS

The initial proposal for the the position of the walkways was close to the slow shoulder separated by a concrete New Jersey Barrier (NJB) which followed the alignment of the N2. The risk of such a position is that the pedestrians will still be in close proximity to the high speed national road traffic, which is not ideal, although separated by a NJB. The cost of the use of a new jersey barrier increases costs. The alternative to a NJB is the use of steel guardrails. The risk of utilizing guardrails is the fact that they cannot withstand excessive impact as well as the fact that they do deflect.

Hence it was suggested by SANRAL to accommodate the pedestrians on a completely separate alignment at least a minimum of 9m away from the N2 road edge, however a distance of 13m was able to be achieved to the available space. This would ensure that the pedestrian and vehicle conflict is reduced and reduce the risk of pedestrian accidents and fatalities. This is shown in Figure 9 below.
Concurrently the design for the capacity upgrade of the N2 was also being undertaken. Hence, to reduce abortive work, the designers were advised by SANRAL to place the alignment of the walkways in its ultimate position which will accommodate the widened N2.

The design criteria adopted for universal access NMT is shown in table 4 below (Department of Transport of South Africa, NMT facility Guidelines).

**Table 4. Design Criteria for Universal Access NMT**

<table>
<thead>
<tr>
<th>Facility</th>
<th>Parameter</th>
<th>Accepted Minimum</th>
<th>Recommended Minimum</th>
<th>ADOPTED</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pedestrian walkway</td>
<td>Min width</td>
<td>1.2 m</td>
<td>1.5 m</td>
<td>2.0 m</td>
</tr>
<tr>
<td>Pedestrian walkway</td>
<td>Max gradient</td>
<td>1:15</td>
<td>1:20</td>
<td>1:25</td>
</tr>
<tr>
<td>Pedestrian walkway</td>
<td>Min corner splay</td>
<td>2.2 m</td>
<td>3 m</td>
<td>5 m</td>
</tr>
<tr>
<td>Separation Distance</td>
<td>Total separation: Distance from shoulder beak point</td>
<td>120 km/h – 5 m</td>
<td>120 km/h – 9 m</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>80 km/h – 2 m</td>
<td>80 km/h – 4 m</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>60 km/h – 1 m</td>
<td>60 km/h – 2 m</td>
<td></td>
</tr>
</tbody>
</table>

It was critical that the max gradients, were strictly followed as this would ensure prams, wheel chairs etc. would have no difficulty using the walkway facilities. Due to the low number of cyclists, it was decided that it is not necessary to create a separate facility for them. Hence, a wider walkway cross-section was selected which would adequately accommodate the odd cyclist when the need arises.

One of SANRAL objectives is to also empower financially challenged communities and provide them with adequate training that would allow them to find jobs and sustain themselves in the long term. Hence, concrete was selected as the material to be used for the walkways. Concrete was selected due to the fact that it is a lot more durable and has a longer life span as opposed to an asphalt walkway which was the alternative. It was also envisaged that the construction of the concrete walkways will be much more labour intensive which will allow SANRAL the opportunity to include the local community to participate in the construction of these walkways, which would be beneficial to them both socially and economically.

**PEDESTRIAN FENCING**

It is human nature to always take the shortest route to one’s destination. Unfortunately, in some instances people tend to ignore the risk associated with such an approach as shown in Figure 10 below.
In an attempt to eliminate this desire of people to cross such high trafficked highways, it was proposed that a pedestrian fence be installed as shown in ‘red’ in the layout plans in Figure 9 above.

A 3-meter-high pedestrian security fence was selected to be used in the centre median to prevent pedestrians from crossing the N2. However, in South Africa, we are always faced with the problem of pedestrians either climbing over the fence or alternatively cutting fences to create a hole in order for them to pass through. A picture of a typical section of the security fence is shown below in Figure 11.

It should be noted that various other design specification were also incorporated in order to address corrosion, cutting strength of wire etc. but will not be discussed under this paper. The purpose of Figure 11 above is to merely highlight the measures taken to prevent a person from climbing, by creating small gaps between the horizontal bars less than 5mm, hence not allowing an average person’s finger to fit in between. Furthermore, thicker vertical and alternative horizontal bars were specified in order to make it difficult and hopefully impossible for them to be easily cut.

Once these pedestrian security fences are installed, pedestrians will be accommodated to cross the N2 on a former agriculture overpass which will be converted to a pedestrian bridge. A picture of the bridge is shown in Figure 12 below.

The elevation of the pedestrian bridge is shown in the plan in figure 12 above. Adequate provision will be made to prevent vehicles from accessing the pedestrian structure.
INTERCHANGES (SALT ROCK I/C AND SHAKAS ROCK)

SIDEWALKS OVER BRIDGES

At the Shakas Rock and Salt Rock interchanges the Average Annual Daily Traffic (AADT) for 2017 is 1195 and 1564 respectively, and the pedestrian counts over a 12-hour period in both directions are 2299 and 1246 respectively. This a high volume of pedestrian movements on a high vehicle trafficked interchange.

Currently, at the Salt Rock I/C the pedestrians walk along the shoulder only separated by a yellow line. and at the Shakas Rock I/C the pedestrians are accommodated with a raised sidewalk only on one side of the bridge. Fortunately, to date there has been no report of pedestrian accidents or fatalities. However, regardless of zero number of accidents, the philosophy adopted is to protect all pedestrians and to be proactive and prevent any future fatalities that may occur.

Due to the fact that the solution to Shakas Rock I/C is fairly rudimentary with the addition of raised sidewalks in either direction to accommodate pedestrians. Only the Salt Rock I/C will be discussed in detail in the contents of this paper as the design approach is more complex in nature. A picture of the existing Salt Rock I/C Bridge or cross-road is shown below in Figure 13.

Figure 13. Picture of Existing Bridges of Salt Rock

Furthermore, it should also be noted that these Interchanges form part of the capacity upgrade of the N2 and will most probably be demolished and reconstructed to accommodate the additional lanes for the N2. For the purpose of this paper, this upgrade of the N2 as well as the interchanges will not be discussed but rather noted. The solution adopted to safely accommodate pedestrians across the bridges will be an interim solution, hence, minimum expenditure as far as possible was adopted to reduce abortive costs. Shown below in Figure 14 is the existing cross-section of the Salt Rock I/C bridge.

Figure 14. Existing Cross-Section of Salt Rock Bridge

Three (3) potential solutions were submitted for review by SANRAL. They are discussed below.

Option 1

The typical cross-section of option 1 is shown below in Figure 15.
As shown in Figure 15 above, option 1 proposal was to only provide a solid NJB on one side of the bridge to accommodate the pedestrians. This proposal is not recommended and supported due to the high number of pedestrians utilizing this interchange as a crossing path over the N2. Pedestrians will still be forced to use the right shoulder where no NJB protection is provided and will still be at a risk.

**Option 2**

The typical cross-section of Option 2 is shown below in Figure 16.

As shown in Figure 16 above, option 2 provides a raised curbed sidewalk on both sides of the bridge to accommodate for movement in both directions. This proposal has merit in that it is addressing the issue of accommodating the high volume of pedestrians on both sides of the bridge. With the inclusion of the sidewalks the minimum height of the parapet from the top of sidewalk to top of parapet is less than the required 1m. Hence, a hand rail will have to be retro-fitted in order to achieve the minimum 1m requirement which can easily be achieved. The only concern was that the inclusion of the sidewalk on the cantilevered portion of the deck will increase the dead load of the structure, hence the designers were requested to undertake a structural integrity analysis of the bridge to determine if the sidewalks option is viable.

Upon investigation of the integrity of the structure, it was discovered that the cantilever was not designed to accommodate this additional dead load. This complication could be dealt with by casting the sidewalk into the deck and doweling additional reinforcement into the existing deck to support the cantilevered section. This is possible but would have major cost implications.

Furthermore, it was discovered from the structural analysis that the existing parapets do not comply with the current requirements of a 100 KN (kilo-newton) impact load. This means if the parapet is hit by a vehicle there is a great chance that the parapet may not serve its purpose of withstanding the impact and there is a great possibility that the parapet and vehicle might fall onto the N2 highway below. In order to address this serious concern, additional reinforcement is required to be cast in to the deck and linked back to the parapet in order to achieve the required 100kn impact load. This would have major cost implications for interim work on an interchange that will be upgraded within the next 5-year horizon. Hence, Option 3 was proposed in order to address all the safety cost concerns.
Option 3

The typical cross-section of Option 3 is shown below in Figure 17.

![Option 3 - Salt Rock Bridge](image)

Figure 17. Option 3 - Salt Rock Bridge

As shown in Figure 17 above, option 3 provides NJB’s on either side of the bridge in order to accommodate the high-volume pedestrians and as well as offer protection for the pedestrians.

A structural analysis was undertaken to determine if the additional dead load of the NJB would have any adverse impacts on the bridge. Fortunately, because of the position of the NJB not been on the cantilever section of the deck, the existing deck is able to accommodate the additional dead load. Hence, no retrofitting designs to the existing structure is required.

This solution is favoured due to the fact that the existing parapets are substandard regarding the 100kn impact load, this option would create an additional barrier for vehicles to impact prior to hitting the existing parapet, hence reducing the impact force onto the existing parapet.

Traffic signals were also introduced as a further safety measure to reduce the risk of accidents and to reduce the operating speeds of vehicles through the Interchange.

Option 3 was favoured as it addressed all safety concerns and reduced unnecessary cost implications, furthermore these NJB’s will be able to be removed and used elsewhere when the I/C is upgraded hence reducing abortive costs.

- **LAY-BYES**

Another concern at the Interchanges was the attraction of pedestrians on the N2 due to the fact that mini-bus taxis provide lifts to pedestrians to their desired destination. This illegal activity attracts a high number of pedestrians and is a cause of many accidents. This issue was identified at both Shakas Rock and Salt Rock Interchanges.

Irrespective of any law enforcement mitigation method implemented which was previously unsuccessful. It was decided that the formalisation of lay-byes be adopted in order to allow for this operation to occur in a safe and regulated environment. A typical layout plan that is currently adopted at all of these hazardous zones is shown in Figure 18 below.
Shown above in Figure 18 is the philosophy adopted to formalise all lay-byes. This enables the taxis to stop completely of the highway in one of the interchange quadrants, thereby eliminating the risk of pedestrians walking on to the N2 carriageway, reducing the rear end accidents occurring because of taxis slowing down to stop.

This allows for passengers to board the taxis safely and also provides adequate acceleration and deceleration lanes for the taxis to leave and join the highway. These lay-byes will cater for pedestrian’s that would like to commute along the highway.

For pedestrians who would like to commute on the cross-roads, lay-byes are also provided on the cross roads to ensure safety for both pedestrians and vehicle. Formal walkways are also provided to guide pedestrians to and from the lay-byes. This will be adopted at both Salt Rock and Shakas Rock I/C’s.

- **SECURITY WALL BETWEEN UMVOTI TOLL PLAZA AND GROUTVILLE**

Shown in Figure 19 below, is an aerial picture indicating the location of the Security Wall, Pedestrian Fence, Toll Plaza and Groutville I/C.

The major issue on this section of the development of the low income residential suburb that is in close proximity to the N2. People tend cross the N2, as well as trade alongside the N2 as well as at the Mvoti Toll Plaza. Various complaints regarding theft, accidents etc. have been reported in this vicinity. It is of great importance that the N2 be protected from such activities and that the pedestrians be provided with formal crossing points as shown in blue above in Figure 19.
As a solution to safeguard the N2 as well as safe-guard the community, a wall is being designed to prevent direct access onto the N2. A typical drawing of the wall is shown below in Figure 20.

![Figure 20. Security Wall](image)

This wall will serve a dual purpose, it will prevent direct access onto the N2 as well as provide a sound barrier which will reduce the road noise experienced by the community. The intention of this wall is to safe-guard the children of the community as there have been reports of them running onto the highway unsupervised. This wall will definitely reduce the number of pedestrians on the N2, at the same time provision is made to still allow them access to the desired locations using underpasses.

**CONCLUSION**

Pedestrian accidents are symptoms of a non-motorized malfunctioning infrastructure system or disobeying laws of the laws governing roads. To best accommodate pedestrians the solution is to remedy the infrastructure by providing corridors for the pedestrians to function which are independent of the vehicle carriageways. This will allow both vehicles and the vulnerable pedestrians to operate in a harmonious and safe environment.

The N2 pedestrian safety improvement as discussed in the contents of this paper merely showcases one of many pedestrian safety improvement projects that are currently being undertaken in South Africa by SANRAL. Close interaction with the Road Traffic Inspectorate (road traffic police) as well as the Road Traffic Management Corporation is ongoing as SANRAL in the process of creating a proactive rather than a reactive database that will allow one to identify high accident zones early and implement remedial measures.

It should be noted that the improvements adopted on this section of the N2 to assist the pedestrians were of a technical nature to an extent and that is all that can be done from an engineering point of view. However, many accidents occur due to pedestrian behaviour as fatigue, intoxication, recklessness, disobedience, inattentiveness, crime etc. To target these issues SANRAL and South Africa have implemented a lot of awareness campaigns through all media streams, education institutions and sign boards along highways to make the motorists aware of the danger.

In conclusion, South Africa as well as the South African National road Agency has placed road safety of the vulnerable road users at the top of its priority and will endeavour to achieve the goal of reducing the number of accidents by 50% by the year 2020 as set out in the Decade of Action Plan.
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- https://www.iol.co.za/motoring/industry-news/pedestrian-deaths-a-global-concern-1949238
### Keywords:
- school zone
- road safety education
- walkway
- bicycle user
- questionnaire survey

### Abstract:
In Japan, we usually go to elementary school on foot and go to junior high school by foot or by bicycle. The existence of school buses is rare and the community is wide, so it is limited.

In the sources such as the Japan Government, traffic accident casualities of children by age are prominent in seven years old. It seems that the fact that it became an elementary school student and that walking outside is routine led to such a result. In addition, among the 143,110 injured people accident during riding a bicycle, junior high school students and high school students accounted for 23%. Bicycle side negligence accounts for 70%, and compliance with bicycle side rules is a problem.

Our efforts are to promote traffic safety education for primary school and junior high school students and to promote compliance with traffic rules, and to grasp changes in awareness and behavior of children and students through preliminary and subsequent observation surveys and questionnaire surveys.

In 2017, considering changes in the consciousness and behavior of pedestrians and bicycles in the efforts carried out in a district. This effort is the second year and we have also considered some improvements. At a certain point, things that keep junior high school students’ pauses at intersections of school bicycles have increased from 14% to 72%.

Through the case of Japan, I would like to share worldwide experts that educational efforts on safety measures on school roads will increase road safety.
Changes in Risk Awareness and Danger Avoidance Behavior on School Road through Traffic Safety Education; Case of Japan, Kikugawa District of Shunan City

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1 INTRODUCTION

In Japan, commuting to elementary school is usually done by walking. Commuting to junior high school is on foot or by bicycle. This is because there are public schools within walking distance. Therefore, in Japan, the existence of school buses is rare and the use of buses is limited to cases where it is impossible to go to school on foot because the community is too wide.

According to the data of related organizations in the Japanese government, traffic accident casualties involving children are prominent in 7-year-old children (Figure 1.).

It seems that the fact that elementary school students walking outside is routine, has led to such a result. In addition, among the 143,110 people injured in accidents while riding a bicycle, junior high school students and high school students accounted for 23% of the total. It was found that in 70% of cases that there was negligence on the bicycle side, and compliance with bicycle side rules etc. can be said to be a problem.

Our efforts aim to promote traffic safety education for elementary and junior high school students and encourage compliance with traffic rules, and through preliminary and subsequent observation surveys and questionnaire surveys, changes in danger awareness and safety behavior of children and students.

In 2017, a survey considering changes in the consciousness and behavior of pedestrians and bicycles was carried out in two districts of Shunan City, in Yamaguchi Prefecture. This effort is in its second year, and we have also considered some improvements since 2015. At a certain point, instances of junior high school students’ pausing at intersections while riding school bicycles have increased from 14% to 72%.

Through the case of Japan, I would like to share with the world experts that educational efforts regarding safety measures on school roads will increase road safety.

2. CIRCUMSTANCES OF THE JAPANESE COMMUNITY AND SCHOOL ROUTES

2.1 Commuting to Japanese Community and Elementary School

The modern elementary school system in Japan was started based on the dissemination of the academic regulation in 1872. Under this Act, the whole country is divided into large, medium and small school districts and primary education institutions are arranged in the school district. In the whole country, eight college districts and one school district were divided into 32 middle school districts, 256 middle schools were divided, 1 middle secondary school divided into 210 primary school districts, and 53760 primary schools were to be placed.
The first elementary schools were quite varied, from those originating from Terakoya; Private Schools for those utilizing the samurai’s educational institution facilities, such as the former clans. Through this history of consolidation and elimination, these elementary schools have been taken over to the elementary school facilities that followed.

From the 1960s to the 1970s, Japan had a high economic growth period. In urban areas, with the rapid increase of the population, problems such as shortages of elementary school classrooms occurred. Meanwhile, in rural areas the number of children decreased with population decline, and elimination and consolidation of elementary schools were conducted. The Kikugawa Elementary School to be introduced this time is a collection of four elementary schools following the consolidation and closure of this period. Therefore, the elementary school has a wide range of school attractions.

In Japan, population peaked in 2008. After 2008, there has been a decline in Japan’s population. As a result, both in urban areas and rural areas, the number of children is decreasing due to this overall decline. In Shunan City(Figure 2 and 3). Kikugawa district and Katsuma district, which were studied here, are all located in the suburbs of local cities. In these schools, the number of children has stabilized and no significant phenomenon in declining student numbers is seen, but compared with the number of children in the 1980s, it is about half that level.

Next, I would like to explain about commuting to elementary school. In elementary school districts, there are several community units. These have long been used as a unit of community-based athletic meetings, festivals and other community activities.

Commuting routes to elementary schools are sent out by members of the community for every road crossing, and they are watching over when the children go to school. In school units, you can see examples of these people who are reading under designations such as the ‘Guardian Guard Corps’.

In addition, school districts of elementary schools are often used as community units, as a unit for grouping communities, and residents have a mechanism to participate in community activities on a community basis.

As they originated 150 years ago, many modern elementary schools originated from the Terakoya within temples and shrine precincts, since these were also used as community units from the Edo period until modern times.

From the above, in Japan, elementary schools and the community are deeply involved, and the elementary school district functions as a community unit. Members of the community, especially the elderly who retired from work and those aged 65 years and over, have contributed to watching the school routes.

2.2 Commuting to elementary school and individual school commuting

In Japan, children are usually instructed to go through a prescribed school route. At that time, those who live in the neighborhood and decide to go to school together are called "collective school".

In group commuting, regardless of grade level, we gather together with neighboring people and walk along the route to school together. In doing so, children in the upper grades try to lead the children of lower grades. Elementary school students go to school by group commuting in the two districts targeted in this research.

Meanwhile, in areas where city distances are short, such as schools in the center of the city center, there are cases where children do not go to school and go to school route individually.

Figure 2. Location of Yamaguchi Prefecture  Figure 3. Location of Sunan City in Yamaguchi Pref.
2.3 For junior high school students, bicycle school and walking school

For junior high school students, there are two forms of commuting to school. One is to go to school on foot, and the other is to go to school by bicycle. In public schools, although it is very rare, there are cases where school buses for school commuting are used.

For junior high school establishment, there are cases where one junior high school is arranged in one elementary school or one junior high school is arranged in multiple elementary schools. In this research, the one-to-one correspondence is the Kikugawa district.

Junior high school districts are often composed of several elementary school districts, and in general the school district is wide. Therefore, many junior high schools commute to school by bicycle as well as walking.

However, in Japan's current situation, there are few cases where bicycle traffic spaces are being maintained along the route from home to junior high school, even though bicycle parks are prepared at junior high school. In this research, we conducted traffic safety education in instances where the bicycle passage space is not well developed, and trying to find changes in danger awareness and safety behavior of bicycle school students commuting to school by bicycle.

2.4 Definition of school roads

In Japan, elementary school routes are defined as routes that roughly 40 children pass through for school commuting. Therefore, in many cases, school routes that less than 40 people use are not designated as school routes. In this research, less than 40 school routes are also called “school road”.

In this research, we define the space through which children and students travel when going to school as school roads.

3. CHARACTERISTICS OF TRAFFIC ACCIDENTS AT SCHOOLS IN JAPAN

3.1 The number of casualties and injuries of seven-year-old children is prominent in traffic accidents during walking

According to traffic accident statistics of 2015, the number of deaths due to traffic accidents for pedestrians was 1,453, and the number of injured was 56,692. The elderly who are 65 years old or over account for 70% of the dead and 32% of the casualties, so the importance of safety measures for the elderly is widely known.

On the other hand, looking at the age of 19 or less, the death toll is as small as 3%, but the number of injured people is 18%. In traffic accidents during walking, the number one in age-specific casualties protrudes by more than 1,400 at age seven.

In the data in increments of 5 years, the group with the highest number of casualties is not the elderly, The group ranging from 5 to 9 years old had, 4,853 casualties. Looking at the data in increments of one year, the number of 7 year-old children is as high as 1,400.

The characteristics of traffic accidents while a 7 years old child is walking are a 73% occurrence during the day, and when it is combined with the litter character before and after sunset it becomes 93%. The rate of traffic accident for adults (16 to 60 years old) is greatly different, from half the number of accidents, both daytime and night time.

Focusing on the purpose of walking by pedestrians who were involved in accidents, the number of schools decreasing impacts children the most in 6 to 15 years old age range. Accidents during schooling increase from 6 - year - old children entering elementary school.

3.2 Bicycle traffic accidents are heavily damaged by middle and high school students

Meanwhile, according to traffic accident data of Japanese government agencies in 2002, of the 179,582 persons who were injured riding bicycles 40,661 were classified as between 13 and 19 years old. It is characterized by many junior high school students and high school student generation damage.
3.3 Many elementary school students are utilizing sidewalk in local cities

According to the road traffic census figures for 2010, schoolchildren accounted for 80% of the sidewalk users of national highway 2 in this target area, and the sidewalk users of the prefectural road Shinnanyo-Tsuwano line also roughly at the same level.

In local cities, it is a fact that elementary school students occupy most of those using sidewalks. Therefore, traffic safety measures on highways such as national highways and prefectural highways will directly lead to safety measures on school roads.

4. OUTLINE OF OUR EFFORTS

4-1 Location of target area

The position of Shunan City in Yamaguchi Prefecture and the location of Kikugawa district are shown in Figure 3. Yamaguchi Prefecture Shunan City has a population of 150,000 people, an agglomeration center of Japan's leading chemical industry, and is an important port city.
4-2 Efforts by Shunan City, Kikugawa District

In the Kikugawa district, both elementary and junior high school students use the city road along the Tonda River as a school road. A sidewalk of 3.5 m in width is being developed on the same line, along the road improvement of the prefectural road Shinnanyo-Tsuwano line, and a bicycle passage zone is installed through the maintenance section. In the future, bicycle commuters are planning to change so that they go through the Shinnanyo-Tsuwano line, but now during the transitional period, bicycles and pedestrians are mixed in a narrow road space along the river, and traffic between them is maintained by adherence to manners.

Table 1. summarizes the efforts by the year 2017 in the Kikugawa district

<table>
<thead>
<tr>
<th>Fiscal Year</th>
<th>Month</th>
<th>Effort contents</th>
</tr>
</thead>
<tbody>
<tr>
<td>2016</td>
<td>July</td>
<td>July 14th Pre-observation</td>
</tr>
<tr>
<td></td>
<td>Sep.</td>
<td>Traffic safety lecture was given to Kikugawa elementary school 3rd–6th graders</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Traffic safety lecture is given to all grain–level junior high school grade</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Pre- and post-questionnaire surveys conducted before and after the lecture</td>
</tr>
<tr>
<td></td>
<td>Oct.</td>
<td>October 4th Post-observation</td>
</tr>
<tr>
<td>2017</td>
<td>Sep.</td>
<td>September 14th Pre-observation,</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Traffic safety lecture is given to all grain–level junior high school grade</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Pre- and post-questionnaire surveys conducted before and after the lecture</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Traffic safety lecture was given to Katsuma elementary school 3–6th graders</td>
</tr>
<tr>
<td></td>
<td>Oct.</td>
<td>October 5th Post-observation</td>
</tr>
</tbody>
</table>
The issues of the school road in the Kikugawa district as follows.

As a feature of bicycle school students, there are so many that do not stop at intersections, and it is necessary to re-recognize traffic rules and manners in order to improve traffic safety.

Here, using the community school of the junior high school, we confirmed the difference in the consciousness of traffic safety between the student and the guardian and local residents.

4-3 Outline of Initiatives

4-3-1 Survey system

Traffic safety education focusing on traffic space on a common school road was conducted for Elementary School children (3rd – 6th graders) and junior high school students (1st – 3rd graders) in the target area, questionnaire surveys were conducted before and after that, in order to understand changes in danger consciousness and changes in behavior intention.(Figure 4.)

The system of investigation in the target area is shown below.

STEP 1 Preliminary observation of target area, questionnaire survey in advance

STEP 2 Implementation of traffic safety lecture Author serves as lecturer

STEP 3 Follow-up observation of the target area, post-questionnaire survey

<table>
<thead>
<tr>
<th>Select district and Schools</th>
<th>Behavioral intention and Passing position</th>
</tr>
</thead>
<tbody>
<tr>
<td>Before</td>
<td>Questionnaire Survey ⇔ Observation Survey</td>
</tr>
<tr>
<td>↓</td>
<td>Lecture for Road Safety Education</td>
</tr>
<tr>
<td>After</td>
<td>Questionnaire Survey ⇔ Observation Survey</td>
</tr>
</tbody>
</table>

Figure 4. Structure of the Survey.

4-3-2 Content of traffic safety lecture

The author gives a traffic safety lecture for about one hour at each school and makes the students recognize the correct traffic rules. The contents explain the situation of the dangerous part of the school road using actual photographs of the site, the seven rules of traffic safety and the five rules of bicycle safe use.
4-3-3 Survey on the current situation of traffic volume etc. on the school road in Katsuma district

We set up video cameras at each major point of school road in school time zone and observed traffic conditions. At the same time, we measured the number of passing cars, bicycles and pedestrians passing through the observation site. We also checked whether change of passage position is seen before and after.

4-3-4 Pre / post questionnaire

We asked each school to take a questionnaire before and after the traffic safety education lecture. After that, we compiled the questionnaire to grasp the understanding before and after the course attendance and the resulting intention of behavioral change. These questionnaire items for elementary school students are shown in Table 3. And Table 4. shows additional items for junior high school students.

<table>
<thead>
<tr>
<th>Table 3. 7 Safety Road Rules, JAPAN</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>7 Safety Road Rules</strong></td>
</tr>
<tr>
<td>1 Use the sidewalk or stay within the line marked</td>
</tr>
<tr>
<td>2 Keep to the right side of the road</td>
</tr>
<tr>
<td>3 Cross the street using the traffic light intersection</td>
</tr>
<tr>
<td>4 Proceed to cross the street at green light on</td>
</tr>
<tr>
<td>5 Should never cross a street that no pedestrian crossing</td>
</tr>
<tr>
<td>6 Before crossing the street, stop and look in both directions</td>
</tr>
<tr>
<td>7 Keep to the right side of walkway</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 4. 5 Rules for Safe Use of Bicycle, JAPAN</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>5 Rules for Safe Use of Bicycle</strong></td>
</tr>
<tr>
<td>1 Cyclists should ride on road, as vehicle</td>
</tr>
<tr>
<td>2 Should ride on walkway, when only allowance</td>
</tr>
<tr>
<td>3 Ride only the left side of street</td>
</tr>
<tr>
<td>4 Must reduce speed on sidewalks and give pedestrians the right of way</td>
</tr>
<tr>
<td>5 Light at night</td>
</tr>
<tr>
<td>6 Riding double is prohibited</td>
</tr>
<tr>
<td>7 Riding side by side is prohibited</td>
</tr>
<tr>
<td>8 Must obey traffic lights at intersections</td>
</tr>
<tr>
<td>9 Check for safety after coming to a full stop at intersections</td>
</tr>
<tr>
<td>10 Children must wear a bicycle helmet</td>
</tr>
</tbody>
</table>
5. CHANGES IN THE RISK AWARENESS OF CHILDREN AND STUDENTS IN KIKUGAWA DISTRICT

We conducted questionnaires before and after the traffic safety lecture for both Kikugawa Elementary School and Kikugawa Junior High School, and answered "I do not think so" on the question "Do you think that the school road is safe when you go to school?" On the question of danger consciousness, we asked about whether we recognized the items of traffic safety 7 rules and the 5 rules of safe bicycle safe use explained in the lecture, and conducted a survey on traffic safety concerns to confirm change in behavior intention.

Based on the results on the change in danger consciousness at Kikugawa Elementary School, it can be seen that the risk awareness after the lecture is increasing as the grade increases (Table 5. and Table 6.).

Table 5. Think a risk awareness of Kikugawa Elementary School Children, 2016

<table>
<thead>
<tr>
<th>answer</th>
<th>3rd grade KIKUGAWA E.S.</th>
<th>4th grade KIKUGAWA E.S.</th>
<th>5th grade KIKUGAWA E.S.</th>
<th>6th grade KIKUGAWA E.S.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>before</td>
<td>after</td>
<td>before</td>
<td>after</td>
</tr>
<tr>
<td>think</td>
<td>54.8%</td>
<td>56.0%</td>
<td>67.0%</td>
<td>60.2%</td>
</tr>
<tr>
<td>N/A</td>
<td>27.4%</td>
<td>22.6%</td>
<td>19.8%</td>
<td>16.1%</td>
</tr>
<tr>
<td>didn't think</td>
<td>14.3%</td>
<td>13.2%</td>
<td>13.2%</td>
<td>20.4%</td>
</tr>
</tbody>
</table>

Table 6. Think a risk awareness of Kikugawa Elementary School Children, 2017

<table>
<thead>
<tr>
<th>answer</th>
<th>3rd grade KIKUGAWA E.S.</th>
<th>4th grade KIKUGAWA E.S.</th>
<th>5th grade KIKUGAWA E.S.</th>
<th>6th grade KIKUGAWA E.S.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>before</td>
<td>after</td>
<td>before</td>
<td>after</td>
</tr>
<tr>
<td>think</td>
<td>66.7%</td>
<td>62.7%</td>
<td>48.1%</td>
<td>44.6%</td>
</tr>
<tr>
<td>N/A</td>
<td>18.3%</td>
<td>20.3%</td>
<td>32.1%</td>
<td>36.1%</td>
</tr>
<tr>
<td>didn't think</td>
<td>15.0%</td>
<td>16.9%</td>
<td>19.8%</td>
<td>19.3%</td>
</tr>
</tbody>
</table>

Figure 5. Compliance of Seven Road Safety Rules, Kikugawa Elementary School, 2016
Based on the results of Kikugawa Junior High School, since we can see that there is a difference in danger consciousness depending on the grade level, we believe that it is necessary to continually conduct traffic safety education and to investigate whether changes due to progress of the grade can be observed (Table 7. and Table 8.).

Table 7. Think a risk awareness of Kikugawa Junior High School Student, 2016

<table>
<thead>
<tr>
<th></th>
<th>1st grade</th>
<th>2nd grade</th>
<th>3rd grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>answer</td>
<td>before</td>
<td>after</td>
<td>before</td>
</tr>
<tr>
<td>think</td>
<td>75.4%</td>
<td>63.2%</td>
<td>68.1%</td>
</tr>
<tr>
<td>N/A</td>
<td>18.8%</td>
<td>26.5%</td>
<td>27.8%</td>
</tr>
<tr>
<td>didn’t think</td>
<td>4.3%</td>
<td>10.3%</td>
<td>4.2%</td>
</tr>
</tbody>
</table>

Table 8. Think a risk awareness of Kikugawa Junior High School Student, 2017

<table>
<thead>
<tr>
<th></th>
<th>1st grade</th>
<th>2nd grade</th>
<th>3rd grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>answer</td>
<td>before</td>
<td>after</td>
<td>before</td>
</tr>
<tr>
<td>think</td>
<td>58.1%</td>
<td>43.6%</td>
<td>73.9%</td>
</tr>
<tr>
<td>N/A</td>
<td>36.4%</td>
<td>45.5%</td>
<td>18.8%</td>
</tr>
<tr>
<td>didn’t think</td>
<td>5.5%</td>
<td>10.9%</td>
<td>7.2%</td>
</tr>
</tbody>
</table>

As a result of questioning whether we are aware of the 7 rules of traffic safety as a safeguard to transport at Kikugawa Elementary School, "As a pedestrian, pedestrians pass the side opposite the roadway" increased by 23.9%, And there is a big change compared to other items (Figure 5.).

![Pre/Post](image)

Figure 6. Compliance of Seven Road Safety Rules, Kikugawa Junior High School, 2016

As a result of Kikugawa Junior High School, "There are no crosswalks, we do not cross wide roads" and "In pedestrian walking pedestrians pass opposite side of roadway" increased by 23%, compared with other items. There is a big change (Figure 6.).
Also, as a result of asking whether the 5 rules of bicycle safety use are recognized as a thing to protect on the use of the bicycle for students at Kikugawa Junior High School, "Bicycles pass the road principle in principle" and "Only when it is exceptionally allowed to pass through the sidewalk "increased by 25%, 38% compared to the previous survey, and a big change was seen. After the incident, it is understood that there is more than 80% recognition in all items, and traffic awareness on bicycle use is high (Figure 7.).

Figure 7. Compliance of Five Rules for Safe Use of Bicycle, Kikugawa Junior High School, 2016
6. CHANGES IN BEHAVIORAL INTENT AND BEHAVIOR OF CHILDREN AND STUDENTS IN KIKUGAWA DISTRICT

In order to observe the current situation of the traffic environment in the Kikugawa district in the school time zone, we designated seven observation points (Figure 8. and Figure 9.) along the prefectural road Shinnanayo-Tsuwano line and the Tonda River which children and students pass as a school road. A traffic volume survey at these seven conducted from 7-9 AM.

The Kikugawa district has one elementary school vs. one junior high school in the same school district, and many school students pass the same school road even if they go from elementary to junior high school. In particular, the Shinkikugawa bridge side intersection has three observation points because a lot of elementary school students and junior high school students travel from the three directions on the north side and the south side along the Tonda River and the prefectural road side.

Photographs of the observation points are shown in Figure. 18 to Figure. 22 below. Observation points A and B are along the Shinnanayo-Tsuwano line of the prefectural road, and in Figure. 4, the back side (upward direction) is point A and the opposite front side (downward direction) is point B. Observation points C · D · E are located at the crossing of the Shinkikugawa bridge at the river crossing, where the south side along the Tonda River is Point C, the north side is Point D, the side along the Shikikugawa bridge is Point E. The observation points F and G are at an intersection near Kikugawa Junior High School, the Point F side is the right side and the straight side is the Point G.

Figure 8. Percentage of keeping traffic location of Kikugawa District, 2016
7. RESULTS OF INITIATIVES IN KIKUGAWA DISTRICT

Kikugawa elementary school students go to school, and all seem to pass along the right side river which is the correct passage position, as shown in "7 narrow roads pass right" in Traffic Safety 7 Law.

The proportion of Kikugawa junior high school students who are passing through the right side is decreasing which is the correct passage position, as stated in "Five passages on the side of the road" of the bicycle safety use during post observation at Points C and G, more than 80% are being protected.

Many junior high school bicycle students are increasing in number after the incident, but only at point G do they not pass at the correct passage position. As a cause, we think that the width of the road is so narrow that only one car can pass through, and the bicycles can not pass through the correct traffic roadway point, so we think that the bicycle traffic infrastructure is insufficient (Figure. 23). The compliance situation of the correct traffic location is shown in Figure.17.

Figure 9. Point C, Pause the bicycle at the intersection, 2016 and 2017

As a problem of traffic behavior on traffic safety, we observed the situation of compliance with temporary suspension of bicycle school students at Kikugawa Junior High School based on video observation at seven observation points, and as a result of the preliminary survey at the Shinkugawa Bridge side crossing point.

Only the percentage was protected, but in the post-survey the protection increased by 14% to 41%. And then, the percentage was protected, but in the post-survey the protection increased by 45% to 72% (Figure 9.).

Based on this, it was effective to reflect the traffic safety lecture to the actual traffic behavior, but half or more bicycle students have not been making stops at intersections, so to continue to pause in the future.

We believe that it will be necessary to continue traffic safety education on traffic safety measures and to establish traffic signs such as temporary stop signs.

8. INFORMATION SHARING AT SCHOOL MANAGEMENT COUNCIL

The Kikugawa Junior High School Management Council was held twice in July 2017 and February 2018, the first time the questionnaire surveyed up to the graduation research interim presentation and the results report of the observation findings were made. The second time we exchanged opinions on 'dangers on the school road' and 'what seems to be a problem in relation to the bicycle and other transport entities' and group opinions from children / students and local people.

The result summarized by KJ Law, the opinion of children and students was that it felt dangerous at the time of commuting to school, there was also a request such as "I want you to turn on street lights" as a measure. The opinions of local residents were transport manners of children and students, and there were opinions on road development of Shinanyo-Tsuwano Line on the prefectural road.
9. RESOLVED ISSUES

The results obtained from the 2-year survey results in the Kikugawa district are summarized below. First, as a result of conducting questionnaire surveys and observation surveys before and after implementing traffic safety lectures, we were able to observe changes in danger consciousness and behavior intention. However, junior high school students' temporary stops by bicycles and improvements in traffic conditions have not been made. The students have not complied with the desired behavior. For this reason, it is necessary to continually carry out traffic safety education in the future.

Secondly, at the school management council of Kikugawa junior high school, as a result of report on initiatives on traffic safety education and exchange of opinions, there was a clear difference in opinion between the students and local residents. Through exchanging opinions between them, it is necessary to improve the traffic environment.

Based on the above, we will continue to conduct road safety education, encourage students and children to adopt appropriate traffic behavior, and incorporate a network of bicycle traffic spaces corresponding to the improvement progress of the prefectural road Shinnanyo-Tsuwano line. It is necessary to comprehensively restructure the school roads of the Kikugawa district.

10. ACKNOWLEDGEMENTS

Acknowledgement: In order to proceed with this research, we received great cooperation from the people concerned at Kikugawa Elementary School, Kikugawa Junior High School and road administrators (Public Transportation Division, River and National Highway Office, Land, Infrastructure and Transportation Ministry, Yamaguchi Prefecture; Road Construction Division, Yamaguchi Prefecture; and Road Division, Shunan City) and the Board of Education (School Security and Physical Division, Education Bureau, Yamaguchi Prefecture).

This is part of the achievement obtained through the instruction by our laboratory activities to graduate research students of National Institute of Technology, Tokuyama College (Kozo Harada, Keita Fukuda, Mao Harada, Yu Ishiwata, Nana Mine and Yuhei Murata) in the previous year.

It is also part of the achievement of the research made possible by the Sasagawa Scientific Research Grant from The Japan Science Society, 2014. I would like to thank all those who have been involved in this research.

This research is part of what was done in 2016 under the aid of subsidies for scientific research. And this research is part of the results of the Yamaguchi Prefectural Government Collaborative Research Project in 2016 and 2017. I would like to express my appreciation in writing.

11. CITATIONS AND REFERENCES

CITATIONS


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2) Meyama, Naoki, Takayuki Honishi, Yu Ishiwata and Nana Mine(2017): A STUDY ON CHANGES IN RISK AWARENESS AND SAFETY BEHAVIOR OF COMMUTING THROUGH DIFFERENCES IN TRAFFIC SAFETY EDUCATION, BETWEEN TWO COMMUNITIES IN YAMAGUCHI PREFECTURE SHUNAN CITY, JAPAN, 5th IRF Middle East and Northern Africa Regional Congress & Exhibition, October 28-31, 2017, Dubai, UAE
KEYWORDS: Child seat, Child restraint systems, Loose installation, NCRUSS

EXTENDED ABSTRACT:
According to the Center for Disease Control (CDC), road crashes are a leading cause of death among children in the United States. In the United States, 663 motor vehicle occupants aged 12 years and younger were killed in 2015 and over 121,000 injured in 2014. Among the 663 children killed, 35% were unbuckled. Child restraint systems (CRSs) are designed to protect children from harmful movement that may result in the event of a crash. Studies have shown the effectiveness of CRSs in improving child passenger safety. According to a NHTSA report in 2002, when a CRS is properly used, the chance of a child fatality in a crash is reduced by 71%. Over the years, the issue with child protection has shifted from not using any restraint to misusing child seat. Addressing the misuses of child restraint systems (CRSs) is necessary to ensure the effectiveness of CRS. Loose installation was the most common mistakes associated with child safety seats. This study aims to identify parent/caregiver characteristics that will be highly associated with loose installation of CRS. To achieve the objective of this study, survey data from National Child Restraint Usage Special Study (NCRUSS) was used. Among the 4,167 child observations, 105 were not restrained by any means (weighted percentage of 2), and 241 were restrained by seatbelt only (weighted percentage of 4.2). Of the inspected children, 100 were seated in the first row (weighted percentage of 2). Lateral moment of CRS greater than 1 inch is considered as loose installation. Of the inspected front and rear facing CRSs, 34% (weighted) have lateral movements greater than 1 inch. Parents/caregivers above 39 years, with increase in age, the loose installation of CRS increased. Gender does not have any effect on loose installation of CRS, whereas the race and ethnicity of the parents/caregivers have strong association. The mean lateral movement of CRS for White is statistically lower than all other races and Asians are statistically lower than those for African Americans. In addition to identifying groups that could benefit from additional education, this study also identified key locations for implementation of such efforts. Approximately 70% of the parents/caregivers said they learned the need of use of child seat from doctor/nurse/hospital personnel and 49% from the internet. The locations with high penetration need to be leveraged to educate about proper installation of child seats. Additionally, doctors/nurse/hospital personnel can be encouraged to give hands-on instruction for proper installation of child seats. Future studies of interest include identification of other major misuses and conducting follow-up / similar study to see if the vulnerable groups identified in this study to evaluate the effectiveness of educational / enforcement efforts over time and across geographic areas.
Abstract

Based on the sustainability concept of equity, economy, and environment, one can define public transportation as one of the transportation modes that can be economical, environment-friendly, and equitable compared to driving personal vehicles. According to the Federal Highway Administration (FHWA), the travel behaviors on roads and streets had changed by 1.1% (2.7 billion vehicle miles) as of December 2013 as compared to December of the preceding year, 2012. Several factors can change driving behaviors. These include environmental commitment, the compatibility of public transportation, and other smart transportation alternatives.

Considering that younger generations are the future leaders in any countries, educating them as to the benefits and hazards of different transportation modes enables them to contribute to the goal of having a cleaner environment and to the reduction of roadway crashes. Travel Without a License (TWL) aims to educate the younger generation, ages 15-21, in any city worldwide of the benefits of riding public transit based on all three aspects of the sustainability concept. The younger generation targeted by the TWL program includes middle school, high school, and undergraduate students. Yet, any persons who are feeling young in spirit can choose to become members of this program.

TWL will first assess the spatial structure of the particular city to identify the locations of school districts, malls or shopping centers, movie theaters, and other attractive places in the city. Trainings will be offered to each candidate before joining the program. TWL will not only develop the independence of the target group with or without a disability, but also will prepare them to leave their homes when the time comes to leave for college or other endeavors of their lives.

Keywords: youth drivers; public transit; travelers’ safety; sustainability; disabilities; new generation

1. Introduction

Having independence and being able to build one's own life is a desirable goal. However, life can be unpredictable, and developmental disabilities can be inevitable. According to the Centers for Disease Control and Prevention (CDC), about one in six children in the United States has one or more developmental delays or disabilities.

Being unable to drive to reach one's preferred destination at one's chosen time can also be a disadvantage for anyone, namely those in the young generation who have been diagnosed with a disability. Previous studies done by KRC Research and Zipcars have shown that a total of 46% of young people in the age range of 18-34 have tried to reduce their driving behaviors (Davis et al. 2012). Smart mobility has been a lively topic within the transportation industry. How can transportation dependency be overcome with smart mobility -- that is, public transportation? The TWL program attracts the younger people who hope to own their transportation independence and be able to travel to the desired destinations without the need of owning a vehicle or having a driver’s license.
2. Vision

The vision of TWL is to educate young people worldwide, with or without a disability, in the benefits imbedded in riding public transit in their cities to encourage them to become healthier, independent, and economical (HIE).

3. Scope of Travel Without a License

The main objective of this program is to assist with training the young people of the world in riding public transportation to reach their destination independently. However, the founder of this program intends to use one city as a model. The chosen city is Denver, Colorado. Metro Denver's middle schoolers, high school students, and college undergraduates will be the target customers, as shown in Figure 1 below. However, the program will be open to any individual who desires to become a member of TWL. Figure 1 was accessed using the following link (http://www.kristalsellsdenver.com/education/denver-metropolitan-school-district-map/). Figure 1 also summarizes the 18 school districts in the Denver Metro area that the first cohort of this program will tackle. For further information, Table 3 in the appendix contains the names of the 18 school districts.

![Diagram of Denver Metropolitan School Districts](image)

**Figure 1: Denver Metropolitan School Districts Mission Statement**

The TWL program aims to focus on the sustainability concept and to highlight the benefits of riding public transit. The vision is to have an eco-friendly community of young people with or without a disability in any city. The program intends to educate and attract young people worldwide to adopt public transit as their main mode of transportation.
Riding public transportation has benefits that perhaps do not readily come to the fore in people's minds. Riders save financially, socialize, even if passively, and contribute to reducing the toxification of the environment. Parents habitually drive their children to desired destinations oblivious of the fact that their children might not so much as know how to interpret the schedules and maps of the available public transportation modes, namely, train, bus, and subway services. Making the youth aware of the benefits of riding public transportation also adds to helping them get ready for being on their own. This program therefore aims to educate, inspire and challenge the younger generation to adapt public transit.

4. Literature Review

Several public transportation programs worldwide have been introduced to accommodate the younger generations with the hope of making public transportation appealing to them. To reduce or even eliminate alcohol use and impaired driving by youth, the National Highway Traffic Safety Administration (NHTSA) identified nine elements as essential comprehensive efforts. These are licensing strategies, supervision, legislation-based programs, enforcement, adjudication, school-based programs, extracurricular activities, adjudication, and community-based initiatives. Williams (1996) found that drivers between ages 16 and 19 were found to have a crash risk four times higher than that of older drivers. It is indeed advisable to continue motivating young drivers to improve safe driving skills throughout their driving education and training. However, the literature review showed no compelling arguments to support the notion that training is an essential method of decreasing crashes. Good examples are Osga (1980), Satten (1990), Collins (1979), and Mortimer (1984). Jones and McCormac (1989), in their study that focused on the effectiveness of high school driver education, revealed no significant differences in crashes among trained and untrained drivers within the first year of being licensed. Wynne-Jones and Hurst (1984) performed a study in New Zealand that evaluated the collision rates of males and females. The findings of Wynne-Jones and Hurst indicate that training had a negative impact on female drivers, while no effect was found regarding the collision rates of males.

The literature review has suggested that conventional interventions to reduce young people’s accidental crashes are not effective. Perhaps it is time to introduce public transportation programs that are accessible to lessen the number of traffic injuries, deaths and accidents while allowing the younger generation to own their independence and travel across cities without a license. This is where TWL comes in, to educate and train the young generation in the dynamics of riding public transportation to their destinations on their own.

On the other side, Christie (2001) performed an international review of driver training effectiveness to reduce roadway safety. Christie’s findings revealed that driver training cannot be measured as an effective crash countermeasure for all drivers. Christie stated that other tactics, namely graduated licensing and increased supervision of novice drivers, are expected to make more lasting and greater contributions to road safety. A similar study was performed by Senserrick and Haworth (2005) while reviewing international and national young driver training. The findings of Senserrick and Haworth found that the major factor that contributed to young novice drivers’ road crash and injury statistics was their inexperience.

Zhao et al. (2006) focused their study on Ontario’s Graduated Licensing System (GLS) to investigate the impacts of driver education on the risk of collisions. Their authors investigated driving behavior and related factors of students in grades 11 and 12 with a graduated license in seven Ontario schools in 1996 and 1998 in two stages of graduated license with G1 as the first stage and G2 as the second stage. Their findings show that no effective effects were observed in G2, which is considered as all drivers who had successfully passed the second stage of the GLS. Preussner et al. (1998) studied the fatal crash risk of teenage drivers. Preussner et al. found a higher risk for teenage drivers traveling during the day or night with two or more other teenagers as passengers.
5. Community Needs Survey

In the effort to reduce death and injury from traffic crashes, TWL will start by identifying the need for transportation by young people in every neighborhood. Based on the community-needs survey responses, the number of individuals who might benefit from TWL will be identified. The survey will also assess specific information to assist volunteers when training to become members of TWL. Survey questions will focus on figuring out whether the individuals are aware of the benefits of public transportation and identifying those who have never used public transportation and those who are in need of public transportation on a daily basis.

6. Methodology

Before starting the first cohort of TWL, presentations will be given to several schools in the city to find out how knowledgeable the young generation is regarding riding public transportation. Surveys will also be handed to parents at the park-in-ride stations as well as other social hubs that support and/or assist families with transportation for individuals with disabilities to enable them to drive to the desired destinations with the assistance of others.

To better evaluate the Metro Denver area high school districts, the 18 high schools in the district will be contacted with an overview of the program's vision and mission. The first strategy is to survey each school to gather data on each student. This methodology will help the program identify the students who are using public transportation as a second mode to navigate around or outside the city; those who hardly ever use public transportation; and those who need public transportation but cannot use it yet. Tables 1 and 2 in the appendix summarize the action plan of TWL.

Once the program has been advertised to the population of the target city, volunteer recruitment will begin and will continue throughout the year. TWL proposed a strategic approach to attract the young generation and volunteers who can train them to confidently ride public transportation.

Some individuals have learned to overcome as they grow the disadvantage of any disabilities so as not to let them be hindrance to a happy life. As a result, many individuals who have been diagnosed with disabilities have been able to take themselves around without the assistance of others. Indeed, allowing individuals with disabilities to travel with the assistance of loved ones can raise a safety issue that TWL hopes to address, while assuring family and friends that they can at least allow their loved ones with disabilities to become members of TWL. Previous studies and programs have shown that individuals are more successful at grasping the teaching of a new program when trained by an individual who has been identified with a similar disadvantage.

6.1. Training Session I-Theory

This training session will focus on theory and participation. TWL participants will learn to read transit maps, timetables, and schedules. They will also learn to download and utilize transit apps on their phones. TWL participants will also learn some of the most important features of public transit and what some of the signs or logos mean. The goal will be to get the participants ready to use the information when riding public transportation. Assessments in the form of visual quizzes will be used to guarantee that TWL participants have grasped the training materials.

6.2. Practice Training Session II- Riding Public Transit

The second phase of the training session will consist of actual use of public transportation around the city with the participants. Each TWL member will be assisted by another individual to ride public transportation for at least four hours. TWL will discreetly monitor the participants to ensure a high absorption of the training sessions and to ascertain that an individual is ready to ride on his/her own. A quiz will be administered focusing on how to ride public transportation effectively. TWL members will continue
taking the training session II until the trainer, the TWL member, and her/his loved ones feel the TWL member is ready to start using TWL on his/her own. Upon successful completion of both TWL trainings, participants will receive their TWL member cards and may enjoy riding public transit.

It is the opinion of this writer that safety might be one of the first issues that parents will think of when they hear about TWL. To gain parents’ trust for their children or loved ones to adopt TWL, the founder of the program hopes to investigate further to learn how transportation policy makers can collaborate to be sure that TWL members feel safe when riding public transportation. TWL’s founder plans to request that each TWL card member be tracked and monitored regularly. Parents and guardians of TWL members who have a disability will also be given access to monitor their children or loved ones. The founder’s goals include security priority for TWL members when riding public transportation and making the public aware of how TWL members can be identified when traveling. To be certain that the safety issue will be taken into account, TWL members’ data will be added to the real-time information transit apps and traffic operations centers, but they can be identified only if they share their TWL information with someone.

During the first year of the program, the TWL program will seek to gather parents’ opinions to evaluate and improve TWL with the hope of attracting more participants to adopt TWL when traveling. Tables 1 and 2 in the appendix summarize the TWL action plan.

7. Conclusion

As technology has evolved, new modes of transportation have been introduced worldwide. As transportation engineers, one of our goals is to educate the young generation on various transportation modes that meet all three criteria of the sustainability concept. TWL hopes to recruit all individuals, whether those individuals are disabled or not. Introducing TWL may have a positive impact on each family within any city. Parents will also have extra time to themselves when they have the option of allowing their children or other loved ones to use public transit unaccompanied. Public transit riders will become more aware of different types of disability and how they should treat or accommodate others when riding public transportation. In addition, the younger generations will develop transportation independence and gain their confidence when travelling on their own.

At this present moment, the author can only predict the benefits and findings of the TWL program. The author hopes that TWL will assist in reducing young drivers’ crashes and increasing public transit riders and revenue. TWL also hopes to assist with the obesity problem, since the younger generations will exercise when walking from their houses to the bus or train stop or station and from their destinations to their houses. Last, TWL will also contribute to the 2020 global warming goal of reducing greenhouse gas emissions, because the aim is to attract the young drivers to adopt mass transportation as their daily mode of transportation.

Acknowledgement

The founder of the TWL program would like to express her sincere gratitude to the Transit Alliance representatives for first offering and then allowing her to attend the Transit Alliance spring scholar of the year 2014 cohort. Additionally, the founder wishes to thank the staff members of the University of Massachusetts Transportation Center (UMTC) for their help and encouragement to pursue this idea and continue contributing to the transportation engineering field, to assist with accessible transportation modes or program options that benefit all individuals, especially the younger generation.

Appendix

The following tables summarize the action plan and outcomes of TWL.

Table 1: Overall Travel Without License Action Plan
<table>
<thead>
<tr>
<th>Overall Action Plan Goal (What am I going to do?)</th>
<th>Tactics (How am I going to implement my Action Plan Goal?)</th>
<th>Timeline (When will I accomplish my tactics?)</th>
</tr>
</thead>
</table>
| To develop a program called “Travel Without a License” that will target high school students within the Metro Denver area to adopt public transportation based on the sustainability aspects. Overall, TWL will educate the high school students on the benefits of using public transportation. | • Research the high school districts within the Metro Denver Area.  
• Contact the identified high schools to inform them of the vision and mission of TWL.  
• Survey the students to gather demographics and their use of public transportation.  
• Identify TWL’s target group (s).  
• Give presentations to parents and students in each high school district to educate them on the benefits of riding public transit based on the sustainability aspects.  
• Meet with the local transit agency representatives to give an overview of TWL.  
• Suggest to the local transit agency the idea of providing discounted passes to high school students who have become members of TWL.  
• Recruit students to become members of TWL and possibly receive yearlong discounted passes from the local transit agency upon becoming members. | Summer/Fall 2019  
Early August 2019  
September 2019-December 2019  
TBD  
TBD |
<table>
<thead>
<tr>
<th>Action Plan Outcome(s)</th>
<th>Measure(s) of Success</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dynamic Interdependence of Transportation</td>
<td>- Freedom; high school students can travel within the city using public transit by themselves.</td>
</tr>
<tr>
<td></td>
<td>- Parents will have more free time to spend with their young children.</td>
</tr>
<tr>
<td></td>
<td>- Develop maturity within young high school students.</td>
</tr>
<tr>
<td></td>
<td>- Increase public transit ridership within the Metro Denver Area. Students might also encourage their parents to adopt public transit.</td>
</tr>
<tr>
<td>Involvement/Participation in TWL</td>
<td>- Invite member students to submit feedback or testimony about how TWL has helped them</td>
</tr>
<tr>
<td></td>
<td>- Invite member students to design logos for members to use.</td>
</tr>
<tr>
<td></td>
<td>- Volunteer to teach new members to read public transit maps and schedules and to give them a tour of the station or train/bus route.</td>
</tr>
<tr>
<td>Socializing</td>
<td>- Meet people while waiting for or riding the bus/train.</td>
</tr>
<tr>
<td></td>
<td>- Start working on assignments while sitting on the bus/train.</td>
</tr>
<tr>
<td></td>
<td>- Explore other areas of Colorado when traveling by bus/train.</td>
</tr>
</tbody>
</table>

**Table 3: Denver Metro Area Cities and School Districts**

<table>
<thead>
<tr>
<th>City</th>
<th>School District</th>
</tr>
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<tbody>
<tr>
<td>Arvada</td>
<td>Jefferson County R-1</td>
</tr>
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<td>Arvada</td>
<td>Westminster 50</td>
</tr>
<tr>
<td>Aurora</td>
<td>Adams-Arapahoe 28J</td>
</tr>
<tr>
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<td>Charter School Institute</td>
</tr>
<tr>
<td>Bailey</td>
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</tr>
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<td>Bailey</td>
<td>Platte Canyon 1</td>
</tr>
<tr>
<td>Black Hawk</td>
<td>Gilpin County RE-1</td>
</tr>
<tr>
<td>Boulder</td>
<td>Boulder Valley RE 2</td>
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<td>Brighton</td>
<td>Adams 12 Five Star Schools</td>
</tr>
<tr>
<td>City</td>
<td>District/Institution</td>
</tr>
<tr>
<td>---------------------------</td>
<td>-------------------------------------------</td>
</tr>
<tr>
<td>Brighton</td>
<td>Brighton 27J</td>
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<tr>
<td>Brighton</td>
<td>Charter School Institute</td>
</tr>
<tr>
<td>Broomfield</td>
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<td>Boulder Valley RE 2</td>
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<td>Broomfield</td>
<td>Jefferson County R-1</td>
</tr>
<tr>
<td>Castle Rock</td>
<td>Douglas County RE 1</td>
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Modeling Safety Performance at Non-signalized Intersections in Alabama: A Study Using eGIS Data

ABSTRACT:
To conform to the Federal Highway Administration standards for Highway Performance Monitoring, Alabama Department of Transportation (ALDOT) has developed its own Enterprise Geographic Information System (eGIS) to monitor traffic safety and record crashes. ALDOT’s eGIS data is featured by a routable roadway network that supports two linear referencing methods to report crashes at either a route milepost (for state routes) or link-node (for local roads). Location-referenced crash data from eGIS can be easily visualized on maps to help researchers and practitioners quickly identify locations where traffic safety improvements are needed. The location reference allows for associating crash information with road inventory, such as Model Inventory of Roadway Elements (MIRE), by matching locations. This study creates a unique database that integrates the eGIS crash data and the road inventory database. Using the integrated database, this study performs an in-depth analysis of safety performance at non-signalized intersections in Alabama, to understand the correlates of safety outcomes at those intersections. Specifically, this study employs a Random Parameter Negative Binomial Model to disentangle the relationships between crash frequency and associated factors including traffic volume. The modeling results are useful for developing Safety Performance Functions for non-signalized intersections in Alabama and offer insights to what countermeasures may be effective in terms of reducing crash frequency at non-signalized intersections. For example, results show that improving the lighting condition may reduce nighttime side impact crashes by 2%. More implications are discussed in this study.

In conclusion, to account for the influences of unobserved heterogeneity, random-parameter models are derived by assuming that the estimated parameters vary across observations, usually according to some pre-specified distribution. When a parameter is found to vary significantly across observations model estimation is considerably more complex because a unique parameter for each observation is estimated for the variable in question. The motivation for such models is to account for unobserved heterogeneity across observations or individuals and to facilitate important new insights regarding the underlying data generating process.
Application of Safety Design Principles for Safer, Slower Roundabouts

Keys to Improving Driver Comprehension and Reducing PDO Crashes

by

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ABSTRACT

U.S. and international research confirms that roundabouts provide substantial safety benefits with respect to injury and fatal crashes. However, many multi-lane, high-flow roundabouts in North America are experiencing higher numbers of vehicular property-damage-only (PDO) crashes than models predict. Whereas, others roundabouts similar in design, context and traffic flows are preforming with low and closer-to-predicted numbers of PDOs. Many of the high crash roundabouts exceed 90 PDOs annually (and some up to 150 or more PDOs per year), while other roundabouts similar in context and traffic flows that are experiencing only 15-30 PDOs per year are within expected predictions for crashes based on U.S. crash data.

While safety research and analysis points to the substantial safety benefits derived from roundabouts in terms of injury and fatal crashes, the high occurrence of excessive PDOs is viewed as problematic.

This paper reviews research and practice that indicates that a comprehensive, principles-based design approach is necessary to achieve expected safety performance and avoid high amounts of PDOs. A comprehensive and integrated design approach – that includes operational analysis, safe geometrics, signing, and pavement markings working together as a unified whole – is necessary for a driving experience that results in the expected amount of vehicular PDO crashes.

INTRODUCTION

The field of road safety has endeavored to achieve zero deaths originating in Sweden, FHWA is committed to the vision of eliminating fatalities and serious injuries, as per there strategic plan, which articulates the goal of "working toward no fatalities across all modes of travel". Indeed, the introduction of roundabouts has been revolutionary in reducing fatal and serious injury crashes at high flow intersections.

The substantial safety benefits of roundabouts with respect to fatal and injury crashes has been widely documented both in North America and internationally. The central problem complicating the safety benefits is a wide variation in rates of vehicular property-damage-only (PDO) crashes among roundabouts of similar capacity and similar measured flow.

As the number of multi-lane, high-flow roundabouts has increased in North America, some are experiencing high numbers of PDO crashes. Despite following U.S. design guidelines, some are exceeding 90 PDOs annually; yet other roundabouts similar in context and traffic flows are experiencing expected amounts in the range of 15-30 PDOs per year (Figure 1).

The two predominant crash types occurring at roundabouts are entering-circulating (failure-to-yield) and lane discipline crashes, and they represent the majority of the PDO crashes (1, 2).

Established transportation research supports the use of a multi-disciplinary, comprehensive, principles-based design approach to achieve maximum safety (3, 4, 5). This paper discusses why a comprehensive design approach – to include operational analysis, geometrics, signing, and
pavement marking, working together as a unified whole – is necessary to reduce crashes at roundabouts.

Roundabout design is of paramount importance. Design elements include: speed limit, sight distance, radius, traffic signs, and pavement markings. But these are just elements. Ultimately, good design requires bringing all of the elements together into a coherent whole. Staging information delivery such that a roundabout is easily interpretable by the driver is key. The objective geometry, visual cues, signage and other elements have to be delivered in a way that conforms to the driver's knowledge, experiences, and understanding.

The goal of good design is optimization. Design optimization is about making all the constituent parts work together in a unified way that minimizes crashes while maintaining efficient function. In this review of literature, combined with experience in the field, we have found that optimally-performing roundabouts (i.e., those that experience minimal crash rates), have this in common: a well-integrated, systematic, and multi-disciplinary approach to geometrics, signing, markings, and speed control.

We have found that problematic roundabouts with excessive crashes appear to lack the implementation of safety principles associated with safe geometrics, or may have confusing signage and/or pavement markings, or a combination of these design elements. On the other hand, roundabouts that are performing within anticipated PDO crash ranges include roundabout and roadway geometric and traffic safety principles for geometry, signing and markings. Figure 1 compares PDOs at several multi-lane roundabouts. The left-hand column shows roundabouts with excessive crashes vs. similar roundabouts (right-hand column) with crash rates within expected range, as predicted by U.S. and U.K. crash prediction models (5, 6).
<table>
<thead>
<tr>
<th>Roundabouts with &gt; 85 PDOs per Year:</th>
<th>Roundabouts with &lt; 20 PDOs per Year:</th>
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<tr>
<td>~90 PDOs per year (40k ADT)</td>
<td>Avg. 15 per year (40k ADT)</td>
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<tr>
<td>Region of Waterloo, ON</td>
<td>New Berlin, WI (4 years of data)</td>
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<tr>
<td>~85 PDOs per year (~30k ADT)</td>
<td>Avg. 10 PDOs per year (28k ADT)</td>
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<td>Joplin, MO</td>
<td>Monona, WI (5 years of data)</td>
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<td>~150 PDOs per year (35k ADT)</td>
<td>Avg. 18 PDOs per year (~30k ADT)</td>
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<tr>
<td>Ann Arbor, MI</td>
<td>Waunakee, WI (1st year of data)</td>
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**FIGURE 1** Comparison of annual PDOs at multi-lane roundabouts.
It is common for engineers and research to attempt to attribute performance of a roundabout to separate and easily discernable individual design components; for example, “it’s too big” or “it’s too small;” or to lack of education about how to drive roundabouts. However, research and practice are confirming that poorly performing roundabouts are a function of the arrangement and relationship of many design elements – to include combinations of geometrics, signing and markings, and the speed context of an application (7). More so than individual design components or driver education or lack of familiarity, driver behavior is influenced by the designs they are driving.

Multi-lane roundabout design objectives for safety and operations compete sharply, making adherence to safety criteria and principles more challenging; e.g., speed control criteria versus entry angle criteria. Optimal design requires the synthesis of geometric design to meet traffic operational objectives within available site constraints. Conformance with design guides implies design consistency, but does not ensure optimal safety. NCHRP 672 is a principles-based design guide aimed at achieving the underlying safety design principles (4). However, design flexibility provided by these principles may inadvertently give the wrong impression that this relieves engineers from the understanding of and adherence to proven multi-disciplinary design principles for safety.

LITERATURE REVIEW

U.S. Highway Safety Methodology

U.S. safety research is focused on: understanding the expected numbers of crashes for a given design, traffic volumes, and speed considerations – Safety Performance Functions (SPFs); allowing for predicting crash numbers and frequency for designs; and determining potential contributing crash factors (Accident Modification Factors, AMFs), and/or countermeasures (8).

There are two primary crash prediction models for roundabouts in the Highway Safety Manual (HSM):

1) “Intersection level roundabout crash prediction methodology:” intended to evaluate the safety performance of an existing roundabout relative to other similar roundabouts. This model includes number of legs (3 to 5), number of circulating lanes, number of crashes annually, and the AADT.

2) “Approach level crash prediction methodology:” used to predict three crash types – entering-circulating, existing-circulating and approach crashes, and utilizes both AADT for entering and circulating traffic and geometric parameters including Entry Width, Approach Width, Angle between Arms, ICD and Circulating Width. (4). These equations utilize this information along with an Empirical Bayes (EB) methodology to predict expected crashes or to compare results at an existing roundabout to other similar roundabouts.
Mark T. Johnson

Chiu et al. conducted safety research on approximately 30 roundabouts in Wisconsin and found that Wisconsin roundabouts have better safety performance (intersection level) than aggregated U.S. data reported in the NCHRP 572 model (1,5). The researchers concluded that reasons could be varied, but that the most plausible reason is attributed to “driving behavior, especially in terms of drivers’ familiarity of roundabouts” (1). However, we find the attribution of improved safety record due to primarily driver familiarity questionable. The roundabouts studied were all constructed prior to or by 2008 and often represented the first roundabouts in communities throughout Wisconsin. Also, there is no attribution to design guidance that informed these designs. The Wisconsin Department of Transportation was an early adopter of a principles-based, comprehensive approach to roundabout design anchored in proven safety principles.

An interesting finding of the Wisconsin research was the safety benefits of flared entry roundabouts (1). This finding is consistent with U.K. safety research for flared entry roundabouts. The roundabouts studied addressed driver expectancy and conspicuity through design features for central islands and implemented roadway and roundabout geometrics for safety, and include easily recognized conventions for traffic signing and pavement markings. The roundabouts had standard lane-use signing and marking arrows (versus fish-hooks), consistent line types for circulatory markings, and a single, clear yield line (without shark’s-teeth markings).

**International Safety Research**

Brown states that important experiments on roundabouts were carried out in the U.K. during a 25-year period (9). This comprehensive work on roundabout safety by the U.K. government collected data on 84 roundabouts with an average of more than five years of crash data per site – a total of 431 junction years of crash data (5). These studies sought to understand the principles of roundabout layout to maximize traffic flow and safety simultaneously, and to produce a practical design method for the highway engineer (9). These accumulated results have been implemented and refined in the U.K. with considerable success (9). This research aimed to develop an integrated design approach that related fatal and injury accidents to traffic volumes and the roundabouts’ geometry. By regressing observed crashes against traffic flow and geometry, they found that different types of crashes related to different variables. Among the key variables were:

- traffic volumes
- approach width (V)
- entry width (E)
- entry path curvatures (deflection)
- angle between arms
- diameter (D)

The result was a geometric crash prediction model: a series of equations to predict crash types and injuries based on geometry and traffic flow (9). The objectives of the U.K. study were
to provide insight to the main crash problems and derive relationships between crash frequency, traffic flows, speeds, and geometric design principles to optimize designs and predict mean crash frequencies.

Other variables found to have relevance to roundabout accidents include (5):

- Intersection sight distance – with excessive sight distance having a negative effect on safety
- Inadequate deflection
- Acute (flat) entry angles which encourage merging behavior
- Poorly designed and/or positioned warning and advance directional signing
- Signs and markings
- Cross fall design

The 2010 U.S. Roundabout Guide (NCHRP 672) reflects many of the key U.K. safety criteria and advice related to safety, including angle between arms, speed control, sight distance, and excessive visibility to left (for right-hand drive), and view angle – to mention a few of the primary design elements (4, 5).

Research in Italy set out to determine crash contributory factors at urban roundabouts (7). The research was based on site inspections and analysis performed by a team of specialists with relevant road safety engineering experience. The poorly performing roundabouts were inspected annually from 2004 to 2009. They found that in many crashes road users’ wrong behavior was significantly affected by a combination of roundabout geometric design, markings and signs, which gave wrong indications to the speed context of the application (7).

This research reviewed geometric data and included: inscribed circle diameter, circulating roadway width, entry width, exit width, entry angle, deviation angle, eccentricity, entry radius, exit radius, radius of deflection of the right turn maneuver, and radius of deflection of the crossing maneuver. Findings showed that the most common geometric design factors were related to speed control, entry angles and combinations of geometrics, signing and markings (7). Geometrics were found to be a contributory factor in 60% of all crashes. Pavement markings were a contributory factor in more than 50% of the crashes. And signs were a contributory factor in nearly 50% of the crashes. This research reflects the challenges with associating specific causes and countermeasures with a key finding, stating: “Even though the identification of contributory factors was based on rigorous analyses and on sound road safety engineering experience, it was a subjective task and some degree of uncertainty existed. Many combinations of contributory factors related to markings, signs, and geometric design were associated with angle crashes at entry. Furthermore, it should be noted that each crash was the result of a unique chain of events and that it might not be possible to identify all of the links for each chain” (7).

**Speed-Focused Studies**

Recent studies focus on speed as a surrogate for safety. Chen et al. examined sight distance preclusion as a means of effecting slower speeds (10). A study by Zirkel et al. supports the
importance of speed control, focused on different sight distance parameters related to operating speed data on approaches, and concluded that drivers have a tendency to drive faster when they have a greater range of vision (11). Isbrands et al. also found that speed can increase the risk of injury-producing crashes, especially at intersections where vehicles may be approaching and entering the intersection with high speed differentials (12).

Another speed-focused study by Mandavilli et al. supports previous research regarding speed as a safety factor, and states that “high approach speeds were an important driver crash factor” (13). The study found that increasing the conspicuity of upcoming roundabouts could be achieved through larger "roundabout ahead" and "yield" signs which could reduce speeds by alerting drivers ahead of time, enhanced landscaping of central islands, and reflective pavement markers and yield signs at the entrance to roundabouts (13).

Roundabout research and subsequent design guidelines from the U.K. reference the slowing effect of visibility restrictions. Their guidance states that “excessive visibility to the right (left for right-hand drive) can result in high entry speeds, potentially leading to accidents” (5). U.K. guidance also suggests the potential use of visibility screens at least 2m high to reduce excessive approach speeds, but has limited the scope to only multi-lane roundabouts where the speed limit is greater than 40 mph (5).

APPLICATION OF SAFETY DESIGN PRINCIPLES TO IMPROVE DRIVER COMPREHENSION AND REDUCT PDO CRASHES

1. Operational analysis/laneage requirements
2. Roundabout geometric design principles
3. Signing and marking

1. Operational Analysis/Laneage Requirements

Most safety research indicates that the entering-circulating conflict is a primary contributor to crashes for multi-lane roundabouts (as is traffic volume). Therefore, macro-level safety benefits are derived from limiting the number of entry and circulating lanes to the minimum necessary while still meeting acceptable operational objectives of delay and queues (5).

Operational performance analysis is necessary to avoid over-design (excessive laneage) and to provide appropriate laneage to meet operational objectives for queue and delay for the entire design life. In high-flow multi-lane conditions higher v/c ratios are prevalent, and if the tools used are too conservative and indicate the need for and determination of additional lanes this will effect safety, feasibility, and costs. High traffic flow multi-lane roundabout applications require sophisticated operational analysis (software) tools.

A thorough understanding of the relative strengths and weaknesses' of available operational tools (gap, empirical, micro-simulation) is necessary in high-flow applications, and using multiple tools is often beneficial. Deterministic software tools in use include Sidra, Arcady and Rodel – robust software tools that offer distinct advantages for high-flow, multi-lane
applications over less robust tools such as spreadsheets that incorporate various equations or HCM (14).

The capacity methodology from the U.K.’s Transport Research Lab (TRL) as implemented by both Arcady and Rodel includes “high definition” queuing theory (15). High definition queuing theory allows for accuracy and understanding of queuing and delay at high v/c ratio conditions up to and exceeding 1.0, which can make a substantial difference in lane decisions at high-flow and high v/c ratio conditions. The high definition models use seven “time-dependent” equations to predict queue lengths and delays. Each equation is selected depending on whether the v/c ratio is less than or greater than 1.0, and if the queues are growing, reducing, or stable. This methodology provides for accurate estimates of queues and delays over the full range of v/c ratios, especially for high v/c ratios including those exceeding 1.0. Queues and delay depend on time and the evolution of flow and capacity. These are described fully in TRL Report 909 (16).

HCM and other gap-based methodologies lack this necessary fine-tuning for optimal design outcomes. Gap-based methodologies utilize a “low definition” model that incorporates a single average v/c ratio in a single queuing equation. This simplified time-dependent model gives good queue and delay predictions at lower v/c ratios. But as v/c ratios approach and exceed 0.9, error rates rapidly increase, particularly in predicted queue and delay. This can result in a call for unnecessary laneage, reducing desired safety outcomes. Ultimately, a good, sound, safe roundabout design might be rejected based on faulty output at high v/c ratios.

Thus, it is important to utilize high-definition models. The detailed look at the full spectrum of v/c ratios allows appropriate matching of capacity to demand. Avoiding unnecessary laneage while still meeting operational requirements and objectives allows the designer to explore safer geometric layouts. Most importantly, the driver's decision-making requirements are simplified. Adherence to safety design principles within the context of a junction include: Speed Control (Fast Path criteria), maximization of angle between arms (90-degree angles are optimal), minimize the number of arms (the fewer, the better; double roundabouts are preferable to a single large 5- or 6-leg roundabout).

2. Roundabout Geometric Design Principles

The two predominant PDO crash types in North America are entry-circulating crashes or “failure-to-yield” crashes, representing approximately 50-70% of all crashes, and lane discipline related crashes. A recent roundabout questionnaire by Washtenaw County Road Commission (Ann Arbor, MI) found that over 1/3 of the respondents believe that one ‘merges’ into a roundabout on entry (incorrect), as opposed to waiting for an appropriate gap (correct) (17).

This mistaken merging behavior is thought to be influenced by roundabout entry angles and the messages they send to drivers. Many North American roundabouts have an overly flattened entry angle, well below desired thresholds as indicated in U.K. guidance.

The U.K. safety research found that flat entry (Phi) angles promote incorrect merging behavior, the result of geometries that send incorrect yield messages at entry. The importance of
entry/Phi angle cannot be overstated. It is a major factor in speed control and fast path measurements.

The entry angle (Phi) serves as a geometric proxy for the conflict angle between entering and circulating traffic streams. The U.K.’s TRL determined that entry angle (Phi) for multi-lane roundabouts should be in the range of 20-40 degrees. It is stated that entry angles below 20 degrees force drivers to strain to look over their left shoulders, creating poor view angles that make it difficult to see circulating traffic. These smaller (flatter) entry angles encourage higher entry speeds due to the visual cues promoting ‘merging driver behavior’ versus the desired priority message of ‘yield’ at entry to circulating traffic.

For multi-lane entries with 3 or 4 RT lanes, the Phi for each lane can be very different from the mean Phi for the whole entry, with the outside lanes (3rd or 4th lanes) having a very small unacceptable Phi even when Phi for the other lanes, and for the entry as a whole, is within the accepted range. Therefore, it is important to check Phi for all lanes (Figure 2), illustrating the design changes necessary to adhere to Phi angle for all lanes. The Phi angle for an entry is the angle between the mean path of the entering traffic and the mean path of the circulating traffic.

**Measuring Phi – Two Methods (18)**

There are two methods for measuring Phi:

1. Method 1: The most common for modern roundabout application due to the smaller ICDs with a nearby exit, as the entering traffic first conflicts with the exiting traffic that has diverged from the circulating traffic. In this case, the angle between the two streams is 2Phi, so Phi is the angle/2.

2. Method 2: Used if the mean path of the entering traffic first conflicts with the mean path of the circulating traffic (before exiting traffic has diverged), typical of older large-ICD roundabouts, and therefore not common in North America. The angle between the two streams is Phi (not the 2Phi of method 1).

Along with Deflection, Entry angle, or Phi angle, may also affect entry speed. Speed control is a achieved as a function of the deflection, angle between arms combined with entry/phi angle in a geometric relationship. Neither the fast path nor Phi angle are on their own a controlling criteria as per NCHRP 672 (4). But in combination, they are decisive in influencing safe driver behavior. Thus, it is imperative that the designer adhere to these fundamental, geometric safety design principles in combination with one another. Figure 2 contrasts non adherence geometry with a design which adheres to these fundamental geometric principles that result in a safe roundabout.
FIGURE 2 Application examples of meeting and not meeting primary geometric design principles: Phi angle for all lanes and intersection angle.
3. Signing and Pavement Markings – Driver Messaging and Information Processing

Roundabouts involve high visual and perceptual demands arising from information acquisition and processing requirements. When signing and other roadway information is presented in too compressed a manner or is incongruent for the design or context, driver comprehension is reduced (19).

Therefore, from an information processing perspective, workload demands in some tasks should be reduced by making it easier for drivers to perform these tasks (9). Signs and pavement markings should be designed and located to minimize detection, reading, and processing time, maximize comprehension, and maximize ability to perform tasks of navigation, guidance, and vehicle control (20). The role of the designer is to simplify messaging and make safe travel as intuitive as possible.

Context and system considerations will affect the design and placement of signing. Too little information, information overload, and sign clutter are undesirable. Optimizing signing and pavement markings requires clarity. Line types, weight, and arrangements are all important to minimize detection, reading and processing time and maximize comprehension.

The first examples in Figure 3 illustrate information overload. The ensuing driver confusion actually works against the design goal of speed reduction upon approach to the roundabout. The large system-to-system style overhead (OH) style signs give incongruent messages to drivers. These types of signs cue drivers that this is a high-speed facility, exactly the opposite of the design intent – speed reduction. Moreover, the unnecessary duplication of information forces larger, competing signs but does not increase drivers’ ease of recognition.

A better way is to stage information delivery by separating destination and lane use information. This hews to our information processing principles. Appropriately scaled signing also mutually reinforces the design intent of slower speeds upon approach.

The second set of illustrations in Figure 3 appropriately separate destination from lane use information. The smaller signs and simplified information improve messaging to the driver. There may be instances when combined information is necessary and desirable, but those would be on a case by case basis, versus a standard.
The three case studies presented here focus on existing roundabouts which were experiencing excessive PDO crashes. The results make clear that drivers were facing significant information processing challenges. Contributory crash factors were identified through an in-service design review (safety audit). Driver confusion was suspected as the culprit and mitigating features targeted this issue. In each of these cases the designers took a comprehensive and integrated approach. The operational analysis kept fundamental roundabout design principles front and center. The group settled on low-cost mitigation measures that included: sight preclusions and lane reduction in one project; and signing and marking changes (consistent circulatory markings, standard lane-use assignment arrows for signing and pavement marking) for all three projects. These solutions resulted in dramatic 60-80% reductions in PDO crashes (21, 22, 23).
Case Study #1: N. 14th and Superior Ave. Roundabout, Lincoln, NE

This 2x3-lane roundabout intersection had experienced approximately 120 crashes in its first year of operation. An in-service design review identified driver comprehension issues and made recommendations to clarify driver information delivery. The mitigation recommendations were implemented with low-cost temporary improvements. Post-modification data showed a 73% reduction in crashes (21, 24).

Issues:
- Failure-to-yield (entering/circulating), side-swipe and rear-end crashes
- Context is a 6-lane suburban arterial roadway (posted speed: 45 mph) 85% speeds ~55 mph therefore higher prevailing speeds
- Low entry (Phi angle) and excessive view angle left
- Lane discipline and priority confusion as to who yields to whom at entry

Recommended countermeasures:
- Lane reductions – based on operational analysis of existing traffic flows, reduce laneage to 2x1 roundabout (reduced entry-circulating conflicts by ~50%)
- Implement standard (and oversized) lane-use signing and marking conventions vs. stylized fish-hook markings.
- Induce speed reduction via preclusion of unnecessary sight distance to left using approaches (6’ high non-see-thru fence)
- Modify existing pedestrian signal to remove resting in green condition.

Results:
73% reduction in crashes (21).

Discussion:
The project team used a comprehensive approach to implementing recommended countermeasures aimed at improving driver comprehension. Each design element mutually reinforced other elements, resulting in less conflicts, improved driver comprehension and a safer roundabout.

Before Changes: Diagrammatical Lane Use Signing and Markings

<table>
<thead>
<tr>
<th>Before Changes: Diagrammatical Lane Use Signing and Markings</th>
<th>After Changes: Conventional Overhead Signing and Markings and Sight-Preclusion Fencing</th>
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<tr>
<td><img src="image1.png" alt="Before Changes: Diagrammatical Lane Use Signing and Markings" /></td>
<td><img src="image2.png" alt="After Changes: Conventional Overhead Signing and Markings and Sight-Preclusion Fencing" /></td>
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</tbody>
</table>
An in-service design review recommended signing and pavement marking changes aimed at improving driver comprehension. The specific crash type targeted was incorrect lane use. The results of these changes were studied by the University of Minnesota Traffic Observatory (22).

**Issues:**
- Lane discipline – outside lane incorrectly circulating, causing crashes with inside lane exiting vehicles, ~ 45%
- Failure-to-yield crashes, ~ 45% (22)

**Recommended countermeasures:**
- Circulatory markings from solid lines and skips through exit replaced with 6” white “consistent/strong” 6’ line, 3’ gap.
- Change from fish-hook to standard style lane-use signing (oversized) and markings.
- Added additional series of two sets further upstream of the roundabout entrance.
- Solid white channelization lane line extended (250’) from yield line, lane line 10’ skip and 10’ gap (150’).
- Replace W11-2 standard “pedestrian crosswalk warning” sign with the R1-6 “yield to pedestrians in crosswalk” sign, with lowered placement height (driver eye level), making signs clearer and more visible to drivers.

**Results:** (22)
- 80% reduction in lane-discipline issues (left turn from outside lane).
- 20% reduction in lane changes at entrance and exits.

**FIGURE 5  Circulating line types: MUTCD 2009 and NCHRP 672.**

Discussion
The original design included the solid-then-skip circulatory markings, as per the Manual on Uniform Traffic Control Devices (MUTCD), and diagrammatical lane-use signage (fish-hook style) and pavement marking arrows (25). The assumption behind implementation of fish-hook style signing and markings was to help prevent the possibility of drivers making wrong-way left
turns. Therefore, it is important to note that the change to conventional lane-use signing and pavement markings produced no evidence of increased wrong-way left turns (22). Similarly, lane discipline was improved with implementation of consistent circulatory markings (22).

<table>
<thead>
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<th>After Changes: Conventional Overhead Signing and Markings</th>
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<table>
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<tr>
<td><img src="image3.png" alt="Before Changes" /></td>
<td><img src="image4.png" alt="After Changes" /></td>
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</table>

**FIGURE 6** Before and after signing and marking changes: diagrammatical vs. conventional.

**Case Study #3: SC 46 and Bluffton Pkwy. Roundabout, Bluffton, SC**

The predominant crash types at this poorly performing roundabout were lane discipline and failure-to-yield at entry crashes. An in-service design review conducted in conjunction with the FHWA Office of Safety and SCDOT’s Office of Safety identified ways to improve the safety record. The review resulted in a comprehensive set of pavement marking recommendations. The goal was to improve driver comprehension via positive driver guidance and optimal driver messaging. The post-modifications crash data showed an 80% reduction in crashes (23).

**Issues:**
- Solid-then-skip circulatory line type reduces driver recognition of correct lane use
- Misalignment from entry to circulating roadway – poor entry angles, confusing priority message (who yields to whom)
- Multiple line types at entry – dotted edge line extended was tangent to down-stream exit leg encouraging merging due to very flat entry angle
Recommended countermeasures:

- Provide a consistent circulating line type (8” white, 6 seg, 3’ gap)
- Realign entry to circulating lane to improve entry alignment and view angles
- 11’ inside circulatory lane width and wider (17”) outside lane width (vs. equal 14’ lane widths)
- Remove shark’s-teeth and dotted edge line extension markings at entry
- Realign placement of new singular and bolder yield line (dotted edge line extended) to clearly delineate yield condition to entry drivers

Results:

80% reduction in crashes (23).

<table>
<thead>
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<th>Before Changes: Misaligned Entry to Circulating</th>
<th>After Changes: Aligned Entry to Circulating</th>
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<td><img src="image1" alt="Image of misaligned entry to circulating lane" /></td>
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<tr>
<th>Before Changes: Confusing line types at entry</th>
<th>After Changes: Clarified priority with heavy yield line</th>
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<tbody>
<tr>
<td><img src="image3" alt="Image of confusing line types at entry" /></td>
<td><img src="image4" alt="Image of clarified entry line" /></td>
</tr>
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</table>

**FIGURE 7** Before and after lane marking changes: misaligned vs. aligned; confusing vs. clarified entry lines.
Discussion

The 2009 MUTCD shows circulatory pavement markings of a solid-then-skip line through the exit area (to allow entry to cross), but these recommendations contradict international guidance for circulatory markings. It is becoming evident through these safety retrofits that the solid-then-skip line type reduces driver recognition of correct lane use, resulting in increased lane discipline crashes. Also the 2009 MUTCD indicates that the yield line is called a dotted edge line extended, which gives the erroneous impression that circulating drivers utilize this line thus promoting tangential placement with the downstream leg.

CONCLUSION

We have found that problematic roundabouts with excessive PDO crash rates appear to have several issues in common, including:

- Entry angle issues: overly flat entry angles, and corresponding excessive view angle to left at entry
- Clarity of message issues: confusing signs and pavement markings that may have excessive amounts of information that are confusing and not consistent with drivers’ expectations
- Design element congruity issues: combinations of design elements that work against the design purpose; e.g., large overhead signs that promote excessive speed at roundabout entry
- Speed/context issues: lacking incorporation of design elements to address speed characteristics, such as conspicuity, safer geometrics, and unnecessarily wide sight distances.

We now have a large body of research and designs in practice which show that comprehensive, integrated roundabout design can keep crash rates negligibly low. This requires a design that adheres to roundabout, roadway, and traffic safety principles. Geometrics, signing and markings anchored in these safety principles, and designed to work one with the other, bring about significant safety benefits.

Roundabout design is principles-based. It is also predicated on design flexibility. Adherence to rigid standards that do not take into account safety and operational outcomes results in a less safe roadway. Of course, this design flexibility does not relieve the engineer from understanding and adhering to proven multi-disciplinary safety design principles. But there must always be an eye toward constant improvement in design. Otherwise, the extraordinary safety benefits of roundabouts — the drastic reduction in death and injury — could be obscured by unnecessarily high PDO crash rates.

Our review of relevant research and state-of-the-art roundabout design practice points to improved driver comprehension via design elements as the key to reducing PDO crash rates at high-flow, multi-lane roundabouts. A comprehensive and integrated design approach is critical. This includes operational analysis, geometrics, signing, and pavement markings — working
together as a unified whole. And each element must be anchored in roadway, traffic and roundabout-specific safety design principles.

REFERENCES


**Innovative Approaches to Curve Delineation for Two-Lane Roads**

Run-off-road crashes are a major problem for rural roads. The average crash rate for horizontal curves is approximately three times that of highway tangents, and run-off-the-road crashes account for 81 percent of fatal crashes occurring at horizontal curves. Two-lane roads in particular tend to be unlit, and drivers may have difficulty seeing or correctly predicting the curvature of horizontal curves. This leads to vehicles entering horizontal curves at speeds that are too high, which can often lead to vehicles running off the roadway. This presentation presents the results of a before-and-after study of passive and active curve delineators.

Three types of passive treatments and three types of active treatments were installed on nine curves on two-lane rural roads. The curves had advisory speed limits between 15 and 35 miles per hour, an average annual average daily traffic (AADT) of 3,000 vehicles, and curve radii between 84 and 580 feet. The active treatments included dynamic curve warning signs, blinking curve warning signs, and sequentially flashing chevrons. Active systems can be activated automatically by in-ground sensors, motion sensors, pavement loop detectors, radar, or ambient light. The passive treatments included retroreflective posts, on-pavement signage, and continuous guardrail deflectors. The research consisted of both a human factors study and an observational study. The human factors study included participants whose speed and lane position were tracked as they drove specially equipped vehicles through each of the curves before and after new treatments were installed. The observational study examined the speed and lane position of traffic on all the curves before and after the installation of the new treatments.

The results of the study were mixed, with every tested system leading to some reductions in speed or encroachments at some parts of the curve while also leading to increases in the same values at other parts of the curve. New or additional treatments to a curve sometimes resulted in an increase of vehicle speed by giving drivers a clearer picture of the curve. Adding retroreflective posts to existing chevrons was effective in reducing the speed and edge line encroachments for traffic in the outer lane, particularly in curves with smaller radii. No clear difference was discovered between passive and active systems or between delineation and warning systems. However, active systems may be more beneficial in a visually complex area in which the warning or delineation system competes for driver attention with other light sources.

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PAPER TITLE: Vehicle Impact Zone of Intrusion on Rigid Barriers

TRACK: TOPIC 3/Towards Zero Deaths

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KEYWORDS: Highway Safety; Rigid Barriers; Full-scale Crash Testing; Zone of Intrusion; and Finite Element Simulations.

ABSTRACT:
Roadways often require roadside barriers to be constructed immediately in front of a hazard (such as a bridge pier), or hazards are placed on top of a barrier (such as a luminaire pole mounted on a barrier). In these situations, an errant vehicle impacting a roadside barrier may contact the hazard located directly behind or on top of the barrier, which may reduce the safety benefits of the barrier. To help mitigate this concern, a zone of intrusion (ZOI) envelope was introduced to determine critical areas above and behind the barriers where hazards should not be placed. The ZOI is a measurement of the farthest protrusion of a critical vehicle component behind the top front corner of a barrier during a crash. Critical vehicle components are those that could snag on the hazard above or behind the barrier and consequently, that contact could pose a risk to the vehicle’s occupants with excessive accelerations or penetration into the occupant compartment. The ZOI envelopes were determined by reviewing full-scale or simulated crash tests for different barrier shapes, heights, and vehicle impacts. The extent that a pickup truck or single-unit truck was determined from full-scale crash tests. In addition, LS-DYNA finite element analysis computer simulations were utilized to investigate the ZOI of commonly installed concrete barriers without crash testing. The development of the ZOI of various rigid barriers, including median barriers and bridge rails, through review of crash testing and computer simulation results, is presented herein.
Vehicle Impact Zone of Intrusion on Rigid Barriers

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1 INTRODUCTION

Roadside barriers are designed to prevent vehicles from impacting hazards located behind the barrier. Roadways often require roadside barriers to be constructed immediately in front of a hazard (such as next to a bridge pier) or have hazards placed on top of a barrier (such as a luminaire pole mounted on a barrier). In these situations, an errant vehicle impacting a roadside barrier risks contacting the hazard located directly behind or on top of the barrier. To help mitigate this concern, a zone of intrusion (ZOI) envelope was developed to determine critical areas above and behind the barriers [1]. The ZOI is defined as the maximum distance that the critical vehicle component protrudes behind the top front corner of the barrier, as shown in Figure 1. Critical vehicle components are those that could snag on the hazard above or behind the barrier and consequently, that contact could pose a risk to the vehicle’s occupants with excessive accelerations or penetration into the occupant compartment. If a ZOI envelope is adequate, any hazard placed outside of the ZOI of a barrier will not pose additional risk to the occupant. Underestimating a ZOI envelope means that the occupant may be injured in the event of a severe impact. Overestimating a ZOI envelope may result in greater costs in accommodating for roadside hazards. The ZOI envelopes vary for different barrier shapes, heights, and test levels.

Figure 1. ZOI (a) Simulated F-shape Barrier [2] and (b) Crash-Tested Vertical-Faced Permanent Concrete Barrier [3]
Ideally, hazards should not be located within the ZOI. If it is necessary to place an object within the ZOI, then it should break away during impact, be moved out of the way by the impacting vehicle such that it poses no harm to the occupants, or be full-scale crash tested. Breakaway and rigid attachments on barriers may penetrate into the occupant compartment or cause excessive vehicle accelerations. Both of these conditions are hazardous to the occupants and may cause a rigid barrier to fail the full-scale crash test safety performance criteria set forth in the National Cooperative Highway Research Program (NCHRP) Report 350 [4]. The ZOI envelopes for various commonly installed rigid concrete barriers determined by either reviewing full-scale crash tests or using LS-DYNA [5] finite element analysis computer simulations were reviewed. In this paper, the development of the ZOI of various rigid barriers, including median barriers and bridge rails, through review of NCHRP Report No. 350 crash testing data as well as computer simulation results, is presented. Moreover, future research needs for the development of updated ZOI envelopes for commonly-installed rigid barriers according to safety criteria set forth in the Manual Assessing for Safety Hardware (MASH), which is an update to NCHRP Report 350 and defines the current roadside safety standards [6, 7] are identified.

2 ZOI STUDIES USING NCHRP REPORT 350 CRASH TESTING DATA

In the late 1990s, a comprehensive review of NCHRP Report 350 full-scale crash testing of bridge rails and median barriers was conducted to identify the extent that a pickup truck and single-unit truck would intrude over the top of a barrier, culminating in the establishment of ZOI envelopes [1]. A field site investigation was also conducted to determine the types of devices commonly attached to traffic barriers. Researchers reviewed video footage of full-scale crash tests using reference geometry to determine the protrusion behind the barriers. From these results, ZOI was developed for various barrier categories, heights, and performance levels, as presented in Table 1. The recommended ZOI guidelines were then included in the AASHTO Roadside Design Guide [8]. ZOI guidelines indicated the spatial zone behind the barrier where vehicle-to-hazard engagement was possible during a real-world crash with impact conditions similar to the design conditions. If barrier attachments or other roadside hardware were located within the ZOI, the hardware may need to be designed to be energy-absorbing or breakaway, and evaluated with full-scale crash testing to confirm crashworthiness. Using the ZOI guidelines and without requiring subsequent crash testing, the minimum lateral offset for rigid attachments placed behind the front vertical face of 813-mm (32-in.) tall, Test Level 3 (TL-3) concrete barriers was 610 mm (24 in.). For the TL-4 conditions, the minimum lateral offset was found to be 2,032 mm (80 in.) and 864 mm (34 in.) for the cargo box and truck cab, respectively.

<table>
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<th>Barrier Class</th>
<th>Barrier Name</th>
<th>Barrier Height mm (in.)</th>
<th>Test Level Equivalence</th>
<th>Vehicle</th>
<th>Maximum Significant Intrusion mm (in.)</th>
<th>Vehicle Component</th>
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Test Level 2

The ZOI for TL-2 barriers is defined by an impact with a 2000-kg (4,409-lb) pickup truck at a speed of 70 km/h (43.5 mph) and at an angle of 25 degrees. This vehicle’s maximum intrusion was found to occur near the impact point where the lower portion of the vehicle crushed inward to allow the hood and the top of the fender to extend over the top of the barrier. For TL-2, there were five applicable crash tests for use in the evaluation, most involving barriers in the 686 to 813 mm (27 to 32-in.) height range. For tests of standard height barriers, the maximum intrusion was found to be near 12 in. [9]. However, one test was conducted on a 508-mm (20-in.) high barrier which resulted in an intrusion of 28 in. [10, 11]. Based on this limited data, the ZOI for TL-2 barriers was defined to be 305 mm (12 in.) for barriers 686-mm (27-in.) high or taller and 711 mm (28 in.) for all shorter barriers, as shown in Figure 2.

![Figure 2. Intrusion Zones for Tall TL-2 Barriers 686 mm (27 in.) and for Short TL-2 Barriers < 686 mm (27 in.)](image)

Test Level 3

Since the TL-3 and TL-4 ¾-ton pickup truck test conditions are the same and TL-4 traffic barriers are often incorporated into areas where TL-3 traffic barriers are acceptable, the pickup truck tests on TL-4 barriers were also used in the process of identifying ZOI’s for TL-3 barriers. A review of nineteen pickup truck tests revealed that, with very few exceptions, the vehicle’s impacting corner intruded the largest extent over the rail. The bumper and front fender of the truck were normally crushed under, and the hood and upper portions of the fender were observed to extend over the top of the barrier. Thus, the greatest extent of intrusion generally occurs early in the impact event while the vehicle’s velocity is very high and its trajectory is still into the barrier. The intrusion of the vehicle’s rear end over the barrier would result in a sideswipe impact that would not be expected to be as severe as the snagging impacts that would occur at the front of the truck.

Barrier height was initially thought to be the variable that would relate best to the intrusion extent. Examination of the intrusion data in comparison to only the barrier height did not reveal this relationship. This finding was attributed to the fact that almost all traffic barriers included in the study had heights within a relatively narrow range from 705 mm (27.75 in.) [12] to 1,067 (42 in.) [13]. Further, the taller barriers generally incorporated sloped concrete faces that tended to lift up the front of the pickup truck, thereby reducing their effective height.

The ZOI was found to be more closely related to barrier class. Traffic barrier classes were combined into three groups based on the size of the intrusion zone. Group one included sloped-faced concrete barriers and steel tubular rails on 6-in. curbs or greater, and their ZOI extends 457 (18 in.) back from the front face of the barrier, as shown in Figure 3. The second barrier group included combination concrete and steel rails, vertical-faced concrete barriers, and all reviewed timber rails, and their ZOI extends 610 mm (24 in.) back from the front face of the barrier, as shown in Figure 4. Group three included steel tubular rails not on curbs or on curbs less than 152 mm (6 in.) high, and their ZOI extends back 762 mm (30 in.) from the front face of the barrier, as shown in Figure 5.
Figure 3. Intrusion Zones for TL-3 Concrete Barriers and Steel Tubular Rails on Curbs

Figure 4. Intrusion Zone for TL-3 Combination and Timber Barriers

Figure 5. Intrusion Zones for TL-3 Steel Tubular Rails not on Curbs
Test Level 4

Single-unit truck tests were found to exhibit greater variation in vehicle behavior than has been observed for the pickup truck tests. Upon impact with the barrier, the test vehicle’s front suspension was usually displaced laterally and longitudinally. During many of these tests, the front wheel on the impact side of the vehicle was pushed under the vehicle. Depending on the height of the barrier and the degree of wheel climb on the barrier’s front face, the vehicle’s front bumper would either crush or ride up over the top of the barrier. The test vehicle’s front fender generally extended well over the top of the barrier, regardless of the action of the bumper and front tire. As the test vehicle was redirected, the rear of the truck typically rotated around and slapped the barrier. Since the bottom of the truck’s cargo box was always higher than the top of the barrier, the rear wheels contacted the barrier instead of the cargo box or frame. The high lateral load on the rear wheels caused both the truck’s cab and its cargo box to roll toward the barrier and generally extend well beyond the front face of the barrier. The extent of roll associated with the truck box and cab was found to vary significantly. Sometimes the two rolled approximately the same amount, while in other cases, the cargo box roll was much greater than that of the cab. Much of the variation in vehicle behavior can be attributed to the torsional rigidity of the truck frame as well as the strength of the cargo box/frame connection. A stiff frame with a good connection would cause the truck cab and cargo box to roll simultaneously. In this case, the windshield and the top of the cab would usually extend well past the front face of the barrier. This behavior exposes the upper corner of the truck cab to an impact with barrier attachments. A more flexible truck frame or a weak cargo box/frame connection allows the cargo box to roll more than the truck cab. In this case, the truck cab generally did not extend far beyond the front face of the barrier. Regardless of the behavior of the truck cab, the cargo boxes were observed to extend well beyond the barrier’s front face. The primary difference between the cab and cargo box intrusions was the height at which they occurred. The maximum intrusion of the cargo box generally occurred at a point well above the top of the barrier, while the truck cab generally intruded near the top of the barrier.

TL-4 barriers tend to be rigid and have a minimum height of 29 in. [14, 15] in order to redirect trucks. Also, most TL-4 barriers are 1,067 mm (42 in.) or less in height [12] since they are not tested with tractor-trailer trucks. Since TL-4 barriers have little height variation, it is not surprising that they exhibited similar intrusion numbers. Therefore, only one ZOI was defined for TL-4 barriers. The TL-4 ZOI is much wider at the top where the cargo box extended significantly past the front face of the barrier. Near the top of the barrier, the ZOI for the single-unit trucks is similar in extent to that of the pickup trucks in the TL-3 analysis. Since there was some variation in the height of the test vehicle’s cargo box, the height of the upper intrusion region was standardized to represent most single-unit truck impacts. The bottom of the intrusion region was placed 229 mm (9 in.) below the top of the barrier, and the top of the region was placed 3 m (10 ft) above the roadway surface, as shown in Figure 6.

![Figure 6. Intrusion Zones for TL-4 Barriers](image)
ZHST STUDIES USING NONLINEAR FINITE ELEMENT ANALYSIS

Due to the high cost of full-scale crash testing, it is not practical to test for the ZOI for every configuration of rigid barrier that exists. Therefore, nonlinear finite element simulation was used to supplement crash testing results to develop ZOI guidelines for desired scenarios. LS-DYNA has been proven to be capable of accurately assessing vehicle barrier interaction for a wide variety of vehicles and impact conditions. This process involved the development of baseline model, simulating impact events, and comparing impact behavior to results observed in full-scale crash tests. Parameter studies were performed to determine how friction and model changes would affect the simulated ZOI. When sufficient confidence in the LS-DYNA modeling accuracy was obtained, the code was used to model recommended barrier design with desired height and shape under desired impact conditions. Model predictions were then used to determine the ZOI using various suspension failure conditions such as tire deflation and joint failures.

ZOI of F-Shape Concrete Barrier

In 2010, MwRSF conducted research into the ZOI of a 1,016-mm (40-in.) high, F-shape concrete barrier when impacted by a 2000P pickup truck under NCHRP Report 350 impact conditions [2]. During this study, it was determined that the front hood geometry would extend over the top of a 1,016-mm (40-in.) tall, F-shape concrete barrier during impact. The ZOI for the barrier impacted by a 2000P vehicle at 100 km/h (62 mph) and at an angle of 25 degrees (NCHRP Report 350 TL-3) was predicted to be 127 mm (5 in.) The ZOI estimate determined during this study was consistent between the various simulation conditions applied to the model, as shown in Figure 7. The ZOI for the barrier impacted by a 2000P vehicle at 72 km/h (45 mph) and at an angle of 25 degrees (NCHRP Report 350 TL-2 standard) was predicted to be between 46 and 64 mm (1.8 and 2.5 in.). Variations were attributed to the quality of the model’s mesh and to the system geometry. The intrusion extent of the 40-in. tall barrier was limited by the front corner of the hood and part of the fender. Due to the limited amount of vehicle intrusion over the barrier, a hazard placed on top of the barrier may not cause additional problems during an impact event.

In this study, parameters including tire-barrier friction, deflation of the tires, and the failure of suspension joints were varied to bracket possible test outcomes. It was found that, in general, increases in tire-barrier friction resulted in increased uplift of the front end of the vehicle during the early portion of the impact. If a tire deflated, there was an immediate loss of load between the vehicle and the barrier. This relaxation of load could affect the vehicle riding up the barrier and the amount of front crush of the vehicle. Suspension failure could affect the loading into the vehicle and significantly affect the overall kinematic behavior of the vehicle throughout the event. For all full-scale test investigations, it was observed that if the suspension joint components had failed, then the tire had also deflated. However, a tire would deflate without causing suspension joint failures. A summary of these parametric evaluations is shown in Figure 7.

Figure 7. Maximum ZOI for 62 MPH, 25-degree impact into 1,016-mm (40-in.) High F-shape Parapet [2]
ZOI of 9.1-Degree Single-Slope Concrete

Few studies have been conducted on the development of the ZOI envelopes of rigid barriers according to the MASH 2009 safety criteria. In a research effort, MwRSF researchers analyzed three Wisconsin DOT 9.1-degree single-slope concrete barriers with top heights of 914 mm (36 in.), 1,067 mm (42 in.), and 1,422 mm (56 in.) [16] for ZOI using nonlinear finite element analysis (FEA). A series of simulations were conducted according to the TL-3 specifications set forth in the MASH, as shown in Figure 8, utilizing a Silverado truck model to compare impact behavior to results observed in full-scale tests. The contact definitions, barrier material model, barrier mesh density, and the timestep were optimized to improve the baseline simulation’s accuracy for the ZOI study. Parameter studies were performed to determine how friction and model changes could affect the simulated ZOI.

![914 mm (36 in.)](image)

![1,067 mm (42 in.)](image)

![1,422 mm (56 in.)](image)

Figure 8. Maximum ZOI Positions with Varied Barrier Heights [16]

Critical ZOIs were evaluated during this research. Each barrier was evaluated under multiple suspension failure conditions, and the maximum values for each barrier height are shown in Table 2. It was assumed that the fender protrusion would not pose a risk to the vehicle’s occupants if impacting a rigid structure located near the ZOI, but an impact with the fender may cause a breakaway structure to dislodge and compromise the integrity of the occupant compartment. Note that the fender protruded 310 mm (12.2 in.) for the 914-mm (36-in.) tall, single-slope barrier, which was the farthest of any of the barriers modeled. This extrusion was less than the current ZOI envelope for a TL-3 concrete barrier.

<table>
<thead>
<tr>
<th>Barrier Height (mm (in.))</th>
<th>Fender ZOI (mm (in.))</th>
<th>Hood ZOI (mm (in.))</th>
</tr>
</thead>
<tbody>
<tr>
<td>914 (36)</td>
<td>310 (12.2)</td>
<td>239 (9.4)</td>
</tr>
<tr>
<td>1,067 (42)</td>
<td>163 (6.4)</td>
<td>165 (6.5)</td>
</tr>
<tr>
<td>1,422 (56)</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Table 2. ZOI and Working Width of the Rigid 9.1-degree Single-Slope Barrier [16]
4 OTHER STUDIES OF RIGID HAZARDS PLACED IN PROXIMITY OF SAFETY BARRIERS

In 2008, MwRSF evaluated barrier attachment [17] with a concentrated effort on luminaire poles. Three NCHRP Report 350 full-scale tests were conducted on an 813-mm (32-in.) tall, 10.8-degree single-slope concrete barrier, with the first two full-scale crash tests evaluating the performance of a luminaire pole placed on top of a single-slope concrete barrier, and the third test evaluating the performance of a luminaire pole placed behind the single-slope concrete barrier. A luminaire pole was mounted on top of the barrier inside the ZOI, as shown in Figure 9.

![Figure 9. Luminaire Pole Attach to Single-Slope Concrete Barrier, Test Nos. ZOI-1 and ZOI-2 (left), and Behind Concrete Barrier, Test No. ZOI-3 (right) [17]](image1)

For the first test (test no. ZOI-1), a 7,985-kg (17,605-lb) single-unit truck (SUT) impacted the barrier system 16.76 m (55 ft) upstream of the centerline of the luminaire pole. The vehicle struck the pole, and the pole was dislodged. Researchers determined that the test was acceptable and that the impact with the luminaire pole did not cause significant risk to the occupants, even though it was within the ZOI envelope. For the second test (test no. ZOI-2), a 2,009-kg (4,430-lb) pickup truck impacted the system 3.35 m (11 ft) upstream of the centerline of the utility pole. The corner of the vehicle hood briefly contacted the pole, and the vehicle redirected away from the system. The test was determined to be acceptable, although the occupant kinematics and injury potential could not be evaluated as no dummy was utilized in the testing. For the third test (test no. ZOI-3), an 8,000-kg (17,637-lb) SUT impacted the single-slope barrier 16.60 m (54 ft – 6 in.) upstream of the centerline of the pole. The pole was within the ZOI enveloped, and the vehicle struck the pole, but there was not enough force to dislodge the pole and no adverse effects were noted on the vehicle due to the impact. Photos from the three tests are shown in Figure 10.

![Figure 10. ZOI Testing – Luminaire Pole on 10.8-Degree Single-Slope Barrier [17]](image2)

Rigid hazards, such as bridge piers, are often found in the vicinity of roadway overpasses both in the medians and along the roadides. There existed a need to develop an economical, reduced length, NCHRP Report 350 TL-3 rigid barrier system for shielding bridge piers and other rigid fixed objects where limited space exists for their placement. MwRSF researchers developed and crash tested a concrete bridge pier protection system that shielded a bridge pier according to NCHRP Report 350 TL-3 criteria [3]. The barrier consisted of an 813-mm (32-in.) tall, vertical-faced permanent concrete barrier with bridge piers located 425 mm (16¾ in.) behind the front face of the barrier, which was within the ZOI envelope. As determined by overhead video analysis, the vehicle hood protruded 503 mm (19.8 in.) beyond the front face of the barrier and struck the bridge pier, as shown in Figure 11. This did not negatively affect the
test, and after deformation of the vehicle hood, the vehicle continued to redirect downstream, despite existence of hazard in the ZOI envelope. Occupant safety was not compromised during the test. Following the successful redirection of the pickup, the safety performance of the stand-alone, vertical concrete barrier was determined to be acceptable.

Figure 11. Bridge Pier ZOI Test [3]

Recently, MwRSF researchers evaluated the safe placement of Illinois Tollway luminaire pole within working width of the Midwest Guardrail System (MGS) [18, 19]. Hazards, including light poles, are not recommended to be placed within the working width of a guardrail system and there are concerns with placing poles in close proximity to guardrail that may affect its ability to safely contain and redirect vehicles. For the standard post spacing MGS, the working width is typically 1,245 mm (49 in.) and is the maximum vehicle protrusion or barrier deflection as measured from the original front face of the guardrail. In a research effort, computer simulation was utilized to select critical impact points, the most severe vehicle-pole-barrier interaction, and critical pole location for the full-scale crash tests. Two crash tests were conducted according to MASH 2016 TL-3 impact safety criteria. In the first test, a 2,267-kg (5,000-lb) pickup truck impacted the combination MGS laterally offset 508 mm (20 in.) in front of a luminaire pole (or 1,041-mm (41-in.) from the front face of the MGS rail) at a speed of 100.7 km/h (62.6 mph) and an angle of 25.2 degrees. The pickup truck was captured and safely redirected while impacting the luminaire pole and disengaging it at the base. In the second test, a 1,097-kg (2,420-lb) small car impacted the MGS laterally offset 508 mm (20 in.) in front of a luminaire pole at a speed of 100.9 km/h (62.7 mph) and an angle 24.8 degrees. The car was safely contained and redirected while minimally contacting the luminaire pole. Thus, a minimum lateral offset of 508 mm (20 in.) between the back of the post and the front face of the breakaway, pole was found to be sufficient to assure a safe performance of the MGS during vehicle impacts without undesired interaction with the pole. Photos of crash tests are provided in Figure 12. Accordingly, guidance was provided for safe pole placement behind the MGS.

Figure 12. 2270P Vehicle Crash Test ILT-1 (left), 1100C Vehicle Crash Test ILT-2 (right)
5 NCHRP REPORT 350 AND MASH ZOI COMPARISONS

In 2009 the American Association of State Highway and Transportation Officials (AASHTO) updated the criteria for the evaluation of roadside hardware beyond the previous NCHRP Report No. 350 standard. The new standard, entitled Manual for Assessing Safety Hardware (MASH), provided updates to test vehicles, test matrices, and impact conditions. In 2016, a new edition of MASH was released (MASH 2016) with few changes to the evaluation of longitudinal barriers and is the current guideline for roadside safety hardware evaluation. The increase in impact severity of MASH 2016 test levels as well as the different geometry of the MASH test vehicles would affect the current ZOI envelopes.

In the previous research, the ZOI was determined for different rigid barriers according to the NCHRP Report 350 TL-2 through TL-5 standards. Based on the combined results of the crash test review and the field investigation, recommendations were made for the safe placement of attachments near traffic barriers under NCHRP Report 350 safety criteria. The recommended ZOI guidelines were then included in the AASHTO Roadside Design Guide, as described early in this paper in Section 2. The barrier systems were crash tested with a 2000P vehicle according to NCHRP Report 350 standards. The updated MASH test vehicle, the 2270P vehicle, has the quarter panel and hood geometrically dissimilar from the 2000P vehicle, as shown in Figure 13. Despite the roll and pitch similarities, the 2270P vehicle showed a significant reduction in the ZOI as compared to the 2000P vehicle, which was directly affected by the geometrical differences in the two frontal profiles. Only preliminary work has been done on evaluating ZOI envelopes with MASH testing and further work is recommended for all barrier heights and shapes.

![Figure 13. 2000P and 2270P Vehicle Front Profile Comparisons](image)

5 CONCLUSIONS AND FUTURE WORK

An extensive literature review of the studies on the development of the ZOI envelopes for various commonly installed rigid concrete barriers was conducted. In these studies, the ZOI envelopes were determined by either reviewing NCHRP Report 350 full-scale crash tests or using LS-DYNA finite element analysis computer simulations. It is recommended that hazards are not placed within the ZOI of a barrier as critical vehicle components may snag on that hazard, which could cause additional occupant risk. However, select full-scale crash tests have been conducted with a hazard placed within the ZOI, which resulted in a successful crash test. Thus, a specific hazard can be placed in the ZOI if has been full-scale crash tested. Additionally, a simulation study was conducted with MASH TL-3 barriers and determined for those particular barrier shapes and heights, that the intrusion was less than current ZOI recommendations. Additionally, it was noted that MASH 2016, the current guideline for roadside safety hardware evaluation, which includes updated test vehicles and impact conditions for crash testing of various roadside safety hardware, could affect the current ZOI envelopes established for NCHRP Report 350 impact conditions. Thus, there exists a need to estimate the ZOI values for each class of barriers according to the MASH 2016. Updated ZOI envelopes for rigid barriers according to the safety criteria set forth in the MASH 2016 will provide state DOTs with a higher confidence level in placing roadside hardware appurtenances near roadside barriers while still providing effective shielding for errant vehicles.
6 ACKNOWLEDGMENTS

Acknowledgment is given to MwRSF personnel for conducting prior research efforts. This work was completed utilizing the Holland Computing Center of the University of Nebraska, which receives support from the Nebraska Research Initiative.

7 REFERENCES


PAPER TITLE: Analysis of Countermeasure Effectiveness in Wire Rope Sections Using ETC 2.0 Probe Data

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AUTHOR (Capitalize Family Name)

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<td>Japan</td>
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KEYWORDS:
road safety
wire rope barrier systems
vehicle behavior
driving sensation

ABSTRACT:
In order to develop a road network under budget and short time constraints, high standard arterial expressways in Japan incorporate four-lane roads with provisional two-way two-lane (hereinafter referred to as TWTL) sections in areas where the traffic volume is low. In the majority of TWTL sections, rubber poles are used to separate inbound and outbound lanes, but the safety risks, including head-on collisions resulting from lane departures of vehicles, are considered problematic. In order to urgently address the situation, wire ropes are now being installed to trial road sections to verify their effectiveness in preventing head-on collisions of vehicles. In order to measure the effectiveness of wire ropes on TWTL sections, this paper uses fluctuations in the number of accidents, video monitoring to detect vehicle positions and driving speed, and questionnaire surveys to determine changes in driving sensation among road users.
Analysis of Countermeasure Effectiveness in Wire Rope Sections Using ETC 2.0 Probe Data

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² Nippon Expressway Research Institute Co., LTD., Tokyo, Japan
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1 INTRODUCTION

On the majority of two-way two-lane (hereinafter referred to as TWTL) expressways in Japan, rubber poles are used to separate inbound and outbound lanes, but the safety risks, including as head-on collisions resulting from lane departures of vehicles, are considered problematic.

On TWTL expressways, four-lane sections or auxiliary lanes are gradually being installed in accordance with traffic conditions. However, due to the urgent need to protect lives, immediate measures to prevent head-on collision accidents are now being demanded.

In order to address the safety of TWTL expressway sections without delay, wire ropes are now being installed to trial road sections with the aim of verifying their effectiveness in preventing head-on collisions of vehicles. The problem is that whether or not this countermeasure will be effective in Japan is unknown.

Also, traffic behavior, including positioning and speed of vehicles, may be affected by wire ropes because they are larger than rubber poles and could have an effect of intimidating drivers. As a similar study, performance verification tests of wire ropes at a test facility have been undertaken by Hirasawa et al.(Hirasawa et al. 2011) but behavior of drivers on expressways have not been observed.

In order to measure the effectiveness of wire ropes in TWTL expressway sections, this paper used fluctuations in the number of accidents, video monitoring to detect vehicle positions and driving speed, and finally a questionnaire survey to highlight changes to the driving sensation among road users.

2 OVERVIEW OF WIRE ROPES

Wire ropes are cable-type barriers used in Europe and America that are generally referred to as "high-tension cable barriers" or "wire rope barriers". An overview of wire ropes and typical rubber poles used in Japan is shown in Table 1.

As shown in the table, wire ropes incorporate an integrated structure of cables and are taller in height than rubber poles. In addition, due to the risk of vehicles being damaged by the rigid material of wire ropes, drivers are expected to drive slower and more carefully through the wire rope sections.

Meanwhile, rubber poles are visually emphasized with a red color scheme.

The widths of the poles (φ) are similar in dimensions: Wire rope poles measure 89.1mm and rubber poles measure 80.0mm.

Table 1. Overview of wire ropes and rubber poles

<table>
<thead>
<tr>
<th>Wire ropes</th>
<th>Rubber poles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Image</td>
<td></td>
</tr>
</tbody>
</table>

Table 1. Overview of wire ropes and rubber poles
3 FLUCTUATIONS IN THE NUMBER OF ACCIDENTS BEFORE AND AFTER THE COUNTERMEASURE WAS INSTALLED

3.1 Roads targeted by analysis

In Japan, wire ropes are installed in the road sections shown in Figure 1.

![Figure 1. Wire rope sections in Japan](image)

3.2 A comparison of the number of accidents

A comparison of the number of accidents before and after the countermeasure was put in place is shown in Table 2.

The results of the analysis confirm the effectiveness in reducing the number of accidents, including only one case of a vehicle crossing over into oncoming traffic, and zero accidents resulting in injuries or fatalities in trial road sections installed with wire ropes. On the other hand, the total number of accidents increased from 45 cases to 112 cases. Compared with rubber pole sections, damage to vehicles is significant when coming into contact with the poles in the wire rope sections. Therefore, it became apparent that hit-and-run accidents were not previously accounted for in the total number of accidents, and the increase in the total number of accidents is inferred to be due to this.
Table 2. Fluctuations in the Number of Accidents

<table>
<thead>
<tr>
<th></th>
<th>Rubber Pole</th>
<th>Wire Rope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Accidents involving crossing</td>
<td>45 cases</td>
<td>1 case</td>
</tr>
<tr>
<td>over into oncoming traffic</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Accidents resulting in</td>
<td>7</td>
<td>0</td>
</tr>
<tr>
<td>fatalities</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Accidents resulting in</td>
<td>6</td>
<td>0</td>
</tr>
<tr>
<td>injuries</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total number of accidents</td>
<td>45 cases</td>
<td>112 cases</td>
</tr>
</tbody>
</table>

*The number of accidents in wire rope sections is from the installation date to October 2017.
*The number of rubber pole accidents is the number of accidents involving collisions with the median in the same interchange sections in 2016.

Also, the results of a comparison of travel speeds in wire rope sections before and after the countermeasure was installed and the t-test results before and after the countermeasure was installed are shown in Table 3–Table 5.

As with changes in spatial travel speed being negligible, t-test results also do not show any significant difference. Therefore, it can be observed that wire ropes and rubber poles do not affect the travel speed of vehicles.

Table 3. Result of t-test for travel speed in wire rope and rubber pole sections (Hokkaido Expressway)

<table>
<thead>
<tr>
<th></th>
<th>Wire rope</th>
<th>Rubber pole</th>
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<tbody>
<tr>
<td>Average value</td>
<td>90.0</td>
<td>90.4</td>
</tr>
<tr>
<td>Variance</td>
<td>14.540</td>
<td>14.601</td>
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<tr>
<td>Number of samples</td>
<td>102</td>
<td>105</td>
</tr>
<tr>
<td>t-value (P-value)</td>
<td>0.671</td>
<td>(0.503)</td>
</tr>
<tr>
<td>Determination result of</td>
<td>No significant difference</td>
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<td>significant difference</td>
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Table 4. Result of t-test for travel speed in wire rope and rubber pole sections (Tokai-Kanjō Expressway)

<table>
<thead>
<tr>
<th></th>
<th>Wire rope</th>
<th>Rubber pole</th>
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<tbody>
<tr>
<td>Average value</td>
<td>81.4</td>
<td>81.9</td>
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<td>Variance</td>
<td>9.083</td>
<td>9.143</td>
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<tr>
<td>Number of samples</td>
<td>98</td>
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<tr>
<td>t-value (P-value)</td>
<td>1.226</td>
<td>(0.222)</td>
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<tr>
<td>Determination result of</td>
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Table 5. Result of t-test for travel speed in wire rope and rubber pole sections (Higashi-Kyushu Expressway)

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<thead>
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<th></th>
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<tr>
<td>Average value</td>
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<td>83.3</td>
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<td>Variance</td>
<td>10.727</td>
<td>10.733</td>
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<tr>
<td>Number of samples</td>
<td>97</td>
<td>101</td>
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<td>t-value (P-value)</td>
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<td>significant difference</td>
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4 CHANGES IN VEHICLE RUNNING POSITION AND TRAVEL SPEED OBTAINED FROM THE VIDEO OBSERVATION SURVEY

4.1 Outline of the video observation survey

A video observation survey was undertaken at 31 locations along high standard arterial expressways in Japan. Video cameras were installed on roadside lighting posts and overpasses in order to capture video footage. The video observation survey locations were selected on the basis of the following conditions:
- Locations evenly distributed across Japan
- Locations with diverse horizontal alignments and longitudinal gradients
- Locations where video observation can be undertaken from a roadside lighting post installed to a road shoulder or overpass

4.2 Analysis of vehicle running position using the video observation survey

Using the video footage captured, the running positions of vehicles at each section (Figure 2) were scanned at 10cm intervals, and a comparison of before and after the wire rope installation was undertaken. In Figure 3 and Figure 4, the horizontal axis shows the average value of vehicle running positions at each section before the installation of wire ropes. The vertical axis shows the scatter diagram indicating an average value of running positions at each section after the installation of wire ropes.

By replacing rubber poles with wire ropes, the running position of both small and heavy-duty vehicles shifted 13cm towards the road shoulder.

Figure 2. Vehicle running position measuring points

Figure 3. Average value of running position at each section before and after implementation (Small vehicles)
4.3 Analysis of vehicle travel speed using the video observation survey

Next, the travel time in two sections was taken from the captured video footage and divided by the distance to calculate the travel speed of each section.

In Figure 5 and Figure 6, the horizontal axis shows the average value of vehicle travel speed before the installation of wire ropes. The vertical axis shows the scatter diagram indicating an average value of travel speed at each section after the installation of wire ropes.

Due to the fact that the average vehicle travel speed before and after the installation of wire ropes is scattered along the 45° line, the vehicle travel speeds at the wire rope section and rubber pole section are almost identical.
5 QUESTIONNAIRE SURVEY OF ROAD USERS TO DETERMINE CHANGES IN DRIVING SENSATION

5.1 Outline of the questionnaire survey

A questionnaire survey was undertaken at 9 parking areas and service areas of high standard arterial expressways in Japan.

The survey was undertaken by fieldworkers verbally asking questions to drivers.

Table 6 shows the list of questionnaire items.

The questionnaire survey locations were selected on the basis of the following conditions:

- Locations evenly distributed across Japan
- Located downstream from wire rope sections or inside wire rope sections
- Locations with large numbers of users

Table 6  List of questions

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<thead>
<tr>
<th>Category</th>
<th>Question</th>
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<tr>
<td>Alternative</td>
<td>Whether countermeasure was perceived</td>
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<td></td>
<td>(Noticed / not noticed)</td>
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<tr>
<td>Five-grade</td>
<td>Sense of security</td>
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<td>evaluation</td>
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<td>Visual influence in right curves</td>
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<td></td>
<td>Ease of driving</td>
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<td></td>
<td>Nervousness</td>
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5.2 Questionnaire results

Figure 7 shows whether the countermeasure was perceived. Figure 8 shows the sense of security. Figure 9 shows the visual influence in right curves. Figure 10 shows the ease of driving. Figure 11 shows the level of nervousness.

Figure 7 shows around 70% of drivers perceived the difference between wire ropes and rubber poles. Figure 8 shows that around 50% of drivers felt a "sense of security" and around 30% felt “neither” insecurity nor security. The "visual influence” shown in Figure 9 indicates that 70% felt no difference, even in right curves where the effect is considered significant. In addition, Figure 10 and Figure 11 show around 60% of drivers answered "neither" to a sense of "intimidation from road width (ease of driving)” or "intimidation from road width (nervousness)”. The figure indicates the majority of road users do not feel any difference between roads fitted with rubber poles and wire ropes.
Figure 7. Whether countermeasure was perceived

Figure 8. Sense of security

Figure 9. Visual influence in right curves

Figure 10. Ease of driving

Figure 11. Nervousness
6 CONCLUSIONS

This paper analyzed fluctuations in the number of accidents, video monitoring to detect vehicle positions and fluctuations in driving speeds, and questionnaire surveys to determine changes in driving sensation among road users. As a result, the effectiveness of wire rope sections in reducing the number of accidents was confirmed. Also, no travel speed analysis using vehicle detector data has identified any difference in driving speed between wire rope sections and rubber pole sections. In addition, the video observation survey indicated the vehicle travel position shifted by 13cm towards the road shoulder and that the vehicle travel speeds have not changed. Furthermore, the questionnaire survey indicates that road users feel an improved sense of security, no negative visual impact, and no changes to their ease of driving or nervousness. Looking at these study results, it has been confirmed that the wire rope sections have no negative influence on drivers, including any effects on their psychological response.

Since the wire rope sections are currently installed only to stretches of surface roads, the development of equipment to be installed on bridge sections of roads shall be necessary to undertake further studies to evaluate their effectiveness.

REFERENCES

Enhance Highway Safety by Using Anti-glare Traffic Signs with Innovative Energy Consumption Free Optical DOTs

Track: Roadside/Median Safety

Author(s)

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Keywords:
Highway Safety, Anti-glare, Traffic Signs, Energy Consumption Free, Optical DOTs

Abstract:
Sun glare is always a problem for road users. When driving into the sun, sun glare can temporarily blind driver’s eyesight. The great brightness of sun glare can also make traffic signs unreadable. A black sign is normally what road users will see under sun glare situation. With this adverse effect, road users can easily miss the information that traffic sign provides, and subsequent accidents may occur. Several measures have been proposed to overcome sun glare problem before, yet not many real practices. In this paper, pilot study on installation of “readable” traffic signs equipped with innovative optical studs is presented. The innovative optical studs were designed and manufactured through academia industrial cooperation to overcome sun glare problem. They were applied to traffic signs in a way similar to those LED illuminated signs. Lab tests show that traffic signs equipped with these newly designed optical studs can give road users clear information under sun glare condition and consume zero energy. On the other hand, information shown in the LED illuminated traffic signs can not be read at all under the same situation. To understand in-situ performance of these innovative traffic signs, the Freeway Bureau, MOTC, Taiwan executed a pilot project using these signs to improve visibility and readability of traffic signs under sun glare condition on national expressway 3 and 6. With in-situ observations, the preliminary results show very promising outcomes. Further modification and study is expected in the near future.
ABSTRACT

Sun glare is always a problem for road users. When driving into the sun, sun glare can temporally blind driver’s eyesight. The great brightness of sun glare can also make traffic signs unreadable. A black sign is normally what road users will see under sun glare situation. With this adverse effect, road users can easily miss the information that traffic sign provides, and subsequent accidents may occur. Several measures have been proposed to overcome sun glare problem before, yet not many real practices. In this paper, pilot study on installation of “readable” traffic signs equipped with innovative optical studs is presented. The innovative optical studs were designed and manufactured through academia industrial cooperation to overcome sun glare problem. They were applied to traffic signs in a way similar to those LED illuminated signs. Lab tests show that traffic signs equipped with these newly designed optical studs can give road users clear information under sun glare condition and consume zero energy. On the other hand, information shown in the LED illuminated traffic signs can not be read at all under the same situation. To understand in-situ performance of these innovative traffic signs, the Freeway Bureau, MOTC, Taiwan executed a pilot project using these signs to improve visibility and readability of traffic signs under sun glare condition on national expressway 3 and 6. With in-situ observations, the preliminary results show very promising outcomes. Further modification and study is expected in the near future.

1 INTRODUCTION

Highway safety relies heavily on traffic signs since traffic signs can guide, warn, and/or regulate road users for better and safer maneuverability. Appropriate installation and maintenance of traffic signs are thus important for giving perceivable and recognizable information to road users in time. Nowadays, reflective traffic signs can exhibit very nice and sharp images with the development of diamond grade reflective sheeting and digital traffic sign printers. Most of the traffic signs work well as expected consequently. Nonetheless, traffic signs installed on east-west highways actually suffer from a daily adverse effect of sun glare. Under sun glare conditions, information shown on traffic signs is pretty much gone due to the eye-blinding sun glare in the background (as shown in Fig. 1). When driving into the sun, sun glare can temporally blind driver’s eyesight. The great brightness of sun glare can also make traffic signs unreadable.
A black sign is normally what road users will see under sun glare situation. Similar situation may also occur when road users driving vehicle in a tunnel toward the exit during daytime. Natural daylight outside a tunnel is generally much brighter than light supplied in the tunnel, which makes road users experience similar situation like sun glare and have difficulty to recognize sign board information (as shown in Fig. 2).

Figure 1. Illustration of the traffic signs under sun glare conditions.

Figure 2. Situation similar to sun glare at the tunnel exit during daytime.

With the sun glare adverse effect, road users can easily miss the information that traffic sign provides, and subsequent accidents may occur. For example, without acknowledging next exit information on the signs from appropriate upstream distance, road users may make sudden multi-line changes when they find out they may miss the right exit they plan to exit, subsequent accidents may thus occur because of this. Duration of sun glare effect on traffic signs depends on geographical location of the signs and the time of a year. Due to the fact that sun glare effect can occur even when the sun is not directly aligned with traffic signs and road user. According to calculations based on longitude, latitude, and azimuth of a traffic sign on certain date, we can know approximately how long sun glare may affect that certain traffic sign. From our calculations, basically 4 to 7 hours daily sun glare effect is expected for most places in the world. It is thus believed that the sun glare problem needs to be solved to enhance highway traffic safety.
2 PRVIOUS IDEAS

Several researchers and some Japanese companies have devoted to solve the reading problem of traffic sign display contents caused by sun glare. Yet, Just a few of them have ever been commercially installed. It may because of ignorance about this problem. It may also because of lack of adequate technique or money. However, these ideas are still worth to evaluate or earn our appreciation. Let’s review these ideas first.

2.1 Partially Cut-Through Technique

This kind of methods uses line-shape, curved line-shape, or dot-shape partially cut-through holes on certain characters or icons of traffic signs. While under sun glare condition, sunlight can passes through these cut-through holes, and thus make important display content of the traffic sign be perceived and recognized by road users. Since the cut-through holes are narrow and the opening area is limited, there won’t be remarkable loss of reflection performance. Moreover, since the structure is not affected by these small cut-through holes, maintenance is basically the same as ordinary traffic signs. According to our survey, line-shape partially cut-through holes of traffic signs may have been tested in Japan for about 15 years (as in figure 3~5) [1]. It can be seen from these figures that cut-through holes do exhibit some characters clearly. Yet it can also be seen in figure 3(a) that characters in the blue sign are partly blocked by the supporting structure (beam of the pillar, reinforcing ribs, or mounting bracket) in its back, which makes the characters in the blue sign unrecognizable or misinterpreted.

One Japanese company tried to overcome the abovementioned structural blocking problem. They proposed custom made solutions, i.e., they design the supporting structure according to the display content so that the best effect can be obtained. The reinforcement ribs will be arranged according to the intended display content during sun glare condition, so that they won’t block the cut-through holes. The rear view diagram will then be drawn accordingly (as in figure 4(b)). The major shortage of this approach is the cost. Specially designed and manufactured signboard may have the cost couple times higher than ordinary boards. The prolonged overall labor hours, human or robot, due to customized rearrangement of supporting structure will also add to the overall cost of a project. How to reduce overall cost and enhance production efficiency is the major challenge for it.

Figure 5 exhibits an example of traffic sign made with dot-shape cut-through holes. As their line-shape siblings, these kinds of signboards are normally applied with reflective sheeting first, then drill holes accordingly. However, cut-through holes on the signboard shown in figure 5 were drilled first then the diamond grade reflective sheeting was applied (as in figure 5(b)), which makes the passing sunlight less dazzling yet prone to dust adhering. To prevent from dust adhered to the exposed reflective sheeting with respective to the cut-through holes locations, clear sheeting may also need to be applied to the back of the signboard. Meanwhile, the supporting structures in the back may also block part of the intended display content and reduce performance of this kind of traffic signs.
It should be noted that the partially cut-through type traffic signs do have drawbacks. One is that passed sunlight may be too dazzling while the sun is right behind the sign. Another one is the amount of sunlight passes through cut-through holes may drop very quickly when the sun moves away from the back of the sign, i.e., when the incident angle increases. Better approaches may be needed to overcome these drawbacks.

![Figure 3. Examples of Japanese line-shape partially cut-through signs](image)

![Figure 4. Supporting structure of the Japanese line-shape partially cut-through signs [1]](image)

![Figure 5. Example of Japanese traffic signs made with dot-shape partially cut-through holes](image)

2.2 Sunlight Softening Technique

Traffic signs adopt sunlight softening technique are generally internal lighting/box type signs. By using a translucent fiber sheet on the back of the chassis and/or some fluffy fibers inside the chassis,
incorporating sunlight into the cabinet, the intended display of the front plate will become brighter and the legibility of display content under sun glare condition will improve dramatically (as in figure 6). As shown in figure 6, the intensity of sunlight reduces due to insertion loss when it transmits through the translucent fiber sheet. The intensity of transmitted sunlight reduces again due to insertion loss when it transmits through fluffy fibers (if any) and the front plate. This mechanism is what we called sunlight softening. This sunlight softening mechanism can prevent road users from overlooking the signs or misreading the route without being able to read until the last minute. The “softened” sunlight can give road users a much more comfortable view of the display content as well. Since generally equipped with fluorescent lamps, this kind of signs can normally be lit up during nighttime and make the sign more eye-catching too.

Some Japanese companies use different approaches. They adopt translucent materials e.g., methacrylic resin board to make signboards. The translucent signboard is then affixed with translucent reflective sheeting. This kind of translucent signboard can also exhibit sunlight-softening behavior and perform as well as the box type signs. Cut-through holes can also be applied to this kind of signboard to enhance its performance under sun glare condition. Figure 7 shows box type (figure 7(a)) and translucent type (figure 7(b)) of sunlight softening traffic signboards manufactured by Japanese companies. Performance of these two examples are both OK but somewhat different.

Figure 6. Illustration of sunlight softening when sunlight transmits through translucent fiber sheets [1, 2]

Figure 7. Two sunlight softening signboards used on Japanese highways [2]
There are other designs using the idea of sunlight softening. Shigeru Miyamoto and Kunimitsu Kobayashi invented a high-luminous-pattern display apparatus and earned the United States Patent 5809681[3]. The abstract of United States Patent 5809681 says “the apparatus includes a surface luminous plate that includes a transparent planar plate stood vertically to a horizontal direction. The transparent planar plate has a front surface and an opposite rear surface, the rear surface being a light incident surface on which direct sunlight and in-air diffused light falls. The surface luminous plate further includes a group of transparent planoconvex lenses arranged uniformly in columns and rows and integrally molded with the front surface of the transparent planar plate. The columns and rows of planoconvex lenses define a front surface of the surface luminous plate, and a planar display plate having a pattern is arranged at the front surface of the surface luminous plate” (as shown in figure 8). With a proper arrangement of the high luminous planoconvex lenses using the abovementioned apparatus, the corresponding traffic sign can be illuminated as planned under sun glare condition. Figure 9 illustrates a speed limit sign equipped with the high-luminous-pattern display apparatus. One can easily tell that the supporting structure of this design consists of many frames and fixtures and thus is very complicated to manufacture and installation process may be very tedious. The sign maybe very heavy and its overall cost may be comparably expansive too. Moreover, there seems no commercial application being reported adopting this apparatus, which may indicate that market value of this system is not that high.

Figure 8. An Explanatory view of the high luminous element and a section view [3]

Figure 9. Explanatory views of the high luminous apparatus and a front view of associated sign [3]
Antonio Alvarez Fernandez-Balbuena et al. at the University Complutense of Madrid presented their study on anti-glare traffic signs by natural light in the *Colour and Light of Architecture* international conference, 2010[4]. They designed a special optical unit composed of two compound parabolic concentrators (CPC) to overcome sun glare problems and the outcome seemed very promising (as seen in figure 10(a)). A compound parabolic concentrator (CPC) [5, 6] is a tilted parabolic profile that can be designed to efficiently collect and concentrate distant light sources, with some acceptance angle and thus can have a high efficiency with good light control (as seen in figure 10(b)). The authors cleverly utilized the light concentration character of CPC. They designed the proposed optical unit in a shape similar to two CPCs with different acceptance angles attached head-on together. With this specific configuration, incident sunlight within acceptance angle will be concentrated at the intersection through the first CPC, the concentrated light will then diffuse through the reversed CPC. With different acceptance angles of these two head-on CPCs, the end-point transmitted sunlight will be much less intense than the incident sunlight, and thus road users can read display content more easily.

![Figure 10](image)

Figure 10. Illustration of the anti-glare system by Antonio Alvarez Fernandez-Balbuena et al [5]

This specially shaped optical unit may have a nice optical property in terms of theoretical anti-glare application on traffic signs. However, commercialization of it may face some challenges. First, it may be very difficult to affix the dual-parabolic shaped unit to a cut-through hole on the signboard. How to affix and tighten it is another. Because traffic signs sometimes are accessible by road users, it is thus important to make sure every affixture of traffic signs are difficult to remove. On the other hand, installation and replacement of these optical units also needs to be handy for crewmembers, otherwise both manufacture cost and maintenance cost will be relatively high.

3. NEWLY DEVELOPED COUNTERMEASURE

After reviewing various ideas proposed by companies and researchers from different countries, we have come up with some preliminary observations about a good sun glare countermeasure for traffic signs.

- Cut-through holes on signboard alone cannot provide good countermeasures since the anti-glare effect drops rapidly when sun moves. Auxiliary measure for collection and concentration of sunlight/ambient light along with cut-through holes can give better results.
• Supporting structures of large traffic signs may block incident sunlight to the back of signboard. As a result, legibility of display content depicted by cut-through holes may reduce. Some Japanese company introduces rearranging support structures solution, yet high overall cost should be taken into account for practical applications.

• Sunlight softening technique can generally give acceptable anti-glare performance. However, the box type signs are a little bit bulky, and the performance seems not as good as the translucent ones. Meanwhile, to replace existing traffic signs with this kind of signboards may cost a lot to do so and is not environmental friendly.

• A great anti-glare traffic sign with complicated design, such as the system in United States Patent 5809681, is just not practically feasible. The countermeasure should have anti-glare performance and easy to manufacture and maintain.

• Any anti-glare countermeasures should keep their display content visible and legible during nighttime. This means that traffic signs with anti-glare countermeasures should look alike ordinary traffic signs as close as possible.

From the above observations, the authors concluded that a good sun glare countermeasure should provide equally good performance regardless sun movement, make only minimum modifications of ordinary signboards, have simple structure and easy to install, fixed tightly and difficult to remove, preserve nighttime visibility and legibility, and be environmentally friendly. With the joint effort of the industrial-academia, a specially shaped optical unit is designed and manufactured to comply with the abovementioned criteria.

The specially shaped optical unit (“the DOTs” hereafter) designed and tested in this study is presented in figure 11(a). Basically, the design does comply with all the considerations discussed above. Meanwhile, the common practice of LED signs is adopted, i.e., the small end of the DOTs can be easily inserted into traditional traffic signs and then convert them into visible and readable ones under sun glare condition yet without any energy consumption. The DOTs gather and concentrate incident light received from the wide rear end then refract into the small front end, thus can deliver traffic sign information clearly to road users traveling under sun glare condition (as in Figure 11(b)). Consequently, the DOTs can enhance traffic safety in adverse visibility situation under sun glare condition. Figure 11(c) shows the front view of a signboard upgraded with the DOTs. It can be seen that there is no major modification of the signboard and the signboard looks very similar to those LED illuminated traffic signs. This implies that no replacement is needed and minimum initial cost can be expected when using the DOTs as sun glare countermeasures for traffic signs.
The optical property of the DOTs is illustrated in figure 12. The incident light acceptable elevation angle is ranging from $0^\circ$ to $+40^\circ$, acceptable azimuth angle is ranging from $-15^\circ$ to $+15^\circ$, whereas the exit light divergence angle is from $-20^\circ$ to $+20^\circ$. It should be noted that when incident light source coming within the zone of $-10^\circ$ to $+10^\circ$ horizontal azimuth, $+12^\circ$ to $+40^\circ$ elevation angle, and horizontal azimuth of emitted light from $-10^\circ$ to $+10^\circ$, the brightest performance can be expected. With such an outstanding optical property, the perceptible horizontal width for road users at 100 meters away from the signboard is up to 72.79 meters theoretically. Moreover, the light extraction efficiency is tested to be above 80%. The DOTs, once installed on signboard, need more than 2 kgf pull out force to remove it. Consequently, the DOTs can perform well even just under ambient daylight or streetlight conditions. Performance of the DOTs under sun glare condition and nighttime streetlight condition are shown in Figure 13(a), (b), respectively.

The DOTs are made of transparent PC material. Test results showed that the material functions normally under ambient temperature $-20^\circ$C ~ $+70^\circ$C without any deformation, crack, or other physical damages. The DOTs were rated IP 54 in the Ingress Protection ratings and remained intact under accelerated
UV test as well, which means the material adopted can resist extreme weather conditions and thus is environmentally friendly. Therefore, we can expect low maintenance cost as well. Considering there is no energy consumption during their time of service, it is reasonable to say that “the DOTs” is a low cost total solution to the sun glare problem.

![Figure 13. Performance of the DOTs upgraded signboard under sun glare & nighttime streetlight conditions](image)

4. THE PILOT PROJECT

The national expressway 6 in Taiwan is basically an east-west highway; meanwhile, some sections of the national expressway 3 are also east west aligned. Road users travel on eastbound of these highway sections suffered from sun glare problems everyday afternoon. The responsible maintenance government agency, the central region branch office, highway bureau, MOTC (CRBO hereafter) thus received many complaint calls about this problem. Knowing the development of the DOTs, CRBO decided to do a pilot project to solve the problem. The aim of the project is to let road users have more reaction time though longer perceptible and legible distance provided by traffic signs upgraded with the DOTs.

![Figure 14. Typical design and installation of signboards used in this project](image)
In this project, two intersections that are under severe sun glare problem and two tunnel exits with nearby downstream intersections are selected for implementing countermeasures. Considering Extruded aluminum board, which is very difficult to drill holes, are required for large traffic signs by law, small size precaution signs are adopted in this project. Typical design and installation of signboards used in this project is shown in figure 14.

Highway engineers and some road users were invited to preliminarily evaluate performance of these signboards. Most of the participants agreed with that these signboards are truly eye-catching. Meanwhile, the perceptible and legible distances averaged from participants’ judgment are about 250m and 150m respectively. These distances do allow road users to have longer and safer weaving sections to do lane changings. The CRBO officers are all satisfied with these preliminary results. Even though it’s very difficult to exhibit the real performance of DOTs by photos, examples in figure 15 and 16 are provided to give some idea about the anti-glare performance of this project. We believe the performance is excellent.

![Figure 15. Performance of anti-glare signboard at a system intersection test site](image)

![Figure 16. Performance of anti-glare signboard at a tunnel exit test site](image)
5. APPLICATIONS

Application of new devices, such as the proposed DOTs, on highway traffic control devices should comply with government regulations. We believe that application of the proposed DOTs to traffic signs can meet the description in part 2 of the manual on Uniform Traffic Control Devices (MUTCD) 2009 edition. Quote from article 2A.07, “New materials and methods can be used as long as the signs and object markers meet the standard requirements for color, both by day and by night”. Application of the DOTs on traffic signs was found able to enhance the sign’s conspicuity and won’t change color of the sign. Meanwhile, article 2A.15 says “Based upon engineering judgment, where the improvement of the conspicuity of a standard regulatory, warning, or guide sign is desired, any of the following methods may be used, as appropriate, to enhance the sign’s conspicuity”. There are 12 measures are then listed following the above paragraph. There are also 6 examples given in figure 2A-1. All of these examples describe measures that can enhance sign’s conspicuity but won’t cause adverse effects. With this in mind, we propose the following traffic sign enhancements with the proposed DOTs (as in figure 17). All little dots shown in signs presented in figure 17 are the proposed DOTs. It can be seen that they all comply with the description of article 2A-15.

![Figure 17 Examples of proposed enhancement of traffic sign’s conspicuity](image1.png)

(a) stripes around a warning sign    (b) plaque above a STOP sign    (c) border line & content enhancement

6. CONCLUSIONS

Sun glare problem is dangerous to road users. Many researchers and companies have proposed various approaches to solve the sun glare problem. Yet each one of those approaches does have something needs improvement. In this paper, a new design of anti-glare countermeasure is proposed and pilot tested. This specially shaped optical unit, the DOTs, are expected to provide equally good performance regardless sun movement, make only minimum modifications of ordinary signboards, have simple structure and easy to install, preserve nighttime visibility and legibility, and be environmentally friendly. Preliminary pilot test results show that design goal is pretty much achieved. Application of the proposed DOTs to traffic signs for
conspicuity enhancement is also considered to comply with MUTCD articles. Nevertheless, further improvement on adaptability of broader application is still needed for this newly proposed product.

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Latent Class Analysis of Human Centered Contributing Factors in Crash Occurrence

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Extended Abstract

Traffic crashes result from a combination of factors related to the road and its environment, vehicles, and road users. A critical component of road traffic crash analyses has been the examination of the driver. Considering that humans assume the higher role in their interactions with the other components of the current road transport system, crash countermeasure implementation should largely be human-centered. Human-centered countermeasures may take the form of improved driver training and testing, education campaigns aimed at changing driving practices, legislation to control driver behavior, and improvements to the design of road systems and automobiles. For this study, latent class analysis was carried out to identify clusters of human-centered causal factors within a large set of heterogeneous crash data. Preliminary data analysis shows that crashes involving some form of driver error (defined to include aggressive driving, failure to yield, following too close, ran traffic control device) made up approximately half of injury crashes. About 44\% of injury crashes were reported to involve women. A third of the drivers involved in injury crashes were unemployed and about 42\% of the drivers were less than 30 years old. Some 9\% of the drivers were under the influence of drugs, alcohol, or medication, while 11\% involved speeding.

The latent class analysis results show that about 2\% of the injury crashes involved drivers believed to be risk takers. This group consisted of male unemployed drivers with no valid driver’s
license and also engaged in drunk driving with no seatbelt. About half of the distracted driving-related injury crashes involved younger drivers and women. Model estimation results also point to high possibility of low seatbelt usage among older drivers. Further, the results indicate that drivers with no employment have high likelihood to drive with no license or invalid driver’s license. In view of these results, recommendations can be made on targeted public awareness and education, backed by comprehensive enforcement programs especially among the identified risk takers group of drivers.
Spatiotemporal Analysis of Traffic Accidents in Kuwait

Towards Zero Deaths

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Traffic accidents, multinomial logit model, location, time-period, Kuwait

Location and time-period can highly be correlated with traffic accidents types. In this study, 287983 traffic accidents that happened in 2013, 2014, 2016, and 2017 were collected from General Traffic Department of Kuwait. The collected traffic accidents occurred in four governorates that included Kuwait-City, Hawally, Al Farwaniya, and Al Ahmadi as those governorates had the highest rate of traffic accidents. The types of traffic accidents that were included in the collected data were crashes, run-over, and rollover accidents. Afterward, the location and the year where and when the accident occurred were chosen to be the independent variable and the dependent variable was the type of accident. Then, a multinomial logit regression model was applied to identify the significant independent variable and the correlations between predictors and the dependent variable. The results showed that both location and time were significant variables that influence the occurring of certain types of accidents. According to the model results, rollover accidents had higher odds of happening in Al Ahmadi governorate. While for the time-period, 2017 was found to have a higher probability of run-over accidents occurring.
Spatiotemporal analysis of traffic accidents in Kuwait

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Abstract

Location and time-period can highly be correlated with traffic accidents types. In this study, 287983 traffic accidents that happened in 2013, 2014, 2016, and 2017 were collected from General Traffic Department of Kuwait. The collected traffic accidents occurred in four-governorates that included Kuwait-City, Hawally, Al Farwaniya, and Al Ahmadi as those governorates had the highest rate of traffic accidents. The types of traffic accidents that were included in the collected data were crashes, run-over, and rollover accidents. Afterward, the location and the year where and when the accident occurred were chosen to be the independent variable and the dependent variable was the type of accident. Then, a multinomial logit regression model was applied to identify the significant independent variable and the correlations between predictors and the dependent variable. The results showed that both location and time were significant variables that influence the occurring of certain types of accidents. According to the model results, rollover accidents had higher odds of happening in Al Ahmadi governorate. While for the time-period, 2017 was found to have a higher probability of run-over accidents occurring.

Keywords: Traffic accidents, multinomial logit model, location, time-period, Kuwait
1. Introduction

Traffic accidents are one of the highest reason for causing death around the globe. According to World Health Organization data (Global Health Observatory 2017), Kuwait had the third highest death rate related to traffic accidents with 18.7 death rate per population compared to other Gulf Co-operation Council (GCC) Countries. For Bahrain, the death rate per population was 8. UAE had a 10.9 death per population and Qatar had 15.2 mortality. While Oman and Saudi Arabia had the highest rate of deaths per population among other GCC countries with 27.4 and 25.4 mortalities respectively. During 2017, 428 deaths and 10219 injuries occurred due to traffic accidents in Kuwait. This requires a study that can provide a sufficient comprehension regarding traffic accidents especially because of the lack of similar studies that can provide similar perception for Kuwait traffic accidents.

Several factors can stimulate the occurrence of traffic accidents including meteorological, temporal, human factors. Alongside these factors, road characteristics can be considered to be another factor related to the occurrence of traffic accidents. A large number of researchers studied those different factors to identify the significant factors and their correlation with traffic accidents in different regions. In addition, road traffic accidents are classified into several types including rear-end collision, side-impact collision, sideswipe collision, dead on collisions, and rollover accidents. In this study, crashes in general including two or more vehicles involved in the accident, run-over, and rollover. First, drivers’ behavior and their tendency to comply with traffic regulations and traffic accidents status were reviewed for the GCC region. Thereafter, run-over and roll over traffic accidents and their potential risk were presented.
In the Kingdom of Saudi Arabia, (Gharaibeh and Abu Abdo 2011) conducted a survey to understand whether youth had a sufficient knowledge regarding traffic safety and regulations. The area of the study was located to be at King Faisal University and the sample was comprised of male drivers only. The results showed that most of the participants were not complying with traffic laws and they were not using the seatbelt. Moreover, important traffic signs were had to be identified by the participants. One of the recommended solutions was aware the drivers about the traffic regulations by TV programs. These programs should be disseminated for the people at an early age. Another research was conducted to determine the influence of using the seatbelt with respect to traffic accidents injuries and if drivers and front-seat passengers comply to the law of using the seatbelt in Saudi Arabia (Bendak 2005). The results were based on a questionnaire survey that was disseminated in two suburbs of Riyadh. Drivers complied more with traffic regulations regarding using the seatbelt than front-seat passengers. The number of injuries due to traffic accidents were decreased after the law was legislated.

( AL-REESI, et al. 2013) prepared a study to find the relationship between motorization rates, economic growth, and traffic accidents in Sultanate of Oman. The study’s findings were based on the national data reported between 1985 and 2009 and by applying several methods to determine the relationships. The results showed that the increase in the economic growth of Oman, increased the rate of motorization which in turn increased traffic accidents and fatalities. (Bener 2005) conducted a study to evaluate the traffic accidents status in Qatar. Findings showed that most of the traffic accidents occurred due to drivers’ careless behavior. Additionally, traffic accidents were ranked the third cause for mortalities in Qatar.
Rollover crashes are responsible for a high portion of fatalities compared to other crash types according to (NHTSA 2010), although their low percentage of occurrence. (Fréchède, et al. 2011) analyzed single-vehicle rollovers that occurred in three Australian states between 2000 and 2007. Rollover crashes were responsible for 35% of the fatalities. Also, they found that there is a significant relationship between crash type distribution and containment of the occupant and dense and urbanized states. Afterwards, (Chen, et al. 2016) studied the factors associated with rollovers and severities by employing classification and regression tree (CART) to identify the significant predictor and support vector machine (SVM) Gaussian radius basis function (RBF) models to evaluate the model performance by utilizing data that were gathered in New Mexico, USA. Results showed that alcohol or drug involvement, seatbelt use, number of travel lanes, driver demographic features, maximum vehicle damages in crashes, crash time, environmental conditions, and crash location were significant variables associated with causing fatalities and incapacitating injuries due to rollover crashes. Consistent with (Chen, et al. 2016), (Dabbour 2017) investigated the factors that were associated with rollover accidents risk for single-vehicle in North Carolina, USA by utilizing a logistic regression model. Dabbour found that vehicle age, speed limit exceeding, rural highways, curved highways, drivers who were younger than 21 years, fatigue, or other medical conditions, and when the light-duty vehicle involved in the collision is not a passenger car increased the risk of rollovers. Similar to the previous study, drunk drivers, or drivers who were influenced by illegal drugs also increased the risk of rollover accidents. According to the study’s findings, the impacts of undivided highways, driver’s gender, and adverse environmental conditions on increasing the risk of rollover accidents were not stable across the chosen time.
Consequently, this study aimed to identify whether the location of the traffic accident and the time when the crash happened are significant variables in Kuwait. Rollover, run-over, and other types of crashes were utilized in this study. Four out of six governorates of Kuwait that had the highest rate of traffic accidents were included: Kuwait City, Hawally, Al Farwaniya, and Al Ahmadi. The timeline for the accidents consisted of four years including 2013, 2014, 2016, and 2017. Thereafter, a multinomial logit regression model was applied to determine the correlation between the independent variables which they are the location of the accident and the year when the accident occurred with the dependent variable which is the type of accident.

The rest of the paper is organized as follows. Section 2 describes the data components and details. The multinomial logit regression model and its results were presented in section 3. Conclusions were presented in section 5.

2. Data Description

The data was obtained from General Department of Traffic that falls under Ministry of Interior of Kuwait. Four intervals time-line were included in this study which they are 2013, 2014, 2016, and 2017. 2015 was not included as the required data was not available. The data for the four years were collected in four different governorates: Kuwait City, Hawally, Al Farwaniya, and Al Ahmadi. A total of 287983 traffic accidents with only considering the three types of traffic accidents that occurred during the above-mentioned time-interval in all the four governorates. All these traffic accidents were considered in this study without excluding any case.

Table 1 is showing the three different traffic accidents types frequencies in the four locations and years. Hawally had the highest total of accidents compared to the other three governorates. While Kuwait City had the second highest crashes followed by Al Farwaniya, then Al Ahmadi.
Noticeably, 242 run-over incidents happened in Al Farwaniya which was the highest number of run-overs. For the roll-overs, Al Ahmadi had the highest rate of roll-over accidents compared to other governorates with 484 roll-overs that occurred in the selected four-years. Observably, general crashes and run-over accidents were higher in the year 2014 than the year 2013. Then, these types of accidents decreased in the following years 2016 and 2017. For rollovers, the accidents counts were almost steady but then decreased dramatically in the year 2017. Each location and year was assigned to a certain code so that it can be easily utilized later to perform the chosen model. Figure 1 showed all types of traffic accidents counts that occurred in the four governorates related to the four years that was chosen in this study. 2016 and 2017 had the lowest accidents counts in all governorates compared to previous years. In contrast, 2014 had higher traffic accidents that occurred in all four governorates. In all the four years, Hawally had the highest accidents rates.

Prior to applying the multinomial logit regression model, multicollinearity between the three independent variables was investigated. The average of variance inflation factor (VIF) was 1.001 which indicates that there was not a multicollinearity issue. As a result, the multinomial logit regression model was chosen in this study to identify the significant variables and determine the correlation between predictors and the dependent variable.

Table 1. Traffic accidents’ types frequencies

<table>
<thead>
<tr>
<th>Location</th>
<th>Year</th>
<th>Abbreviation</th>
<th>Crash</th>
<th>Type of accident</th>
<th>Rollover</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Kuwait City</td>
<td>LC1</td>
<td>88039</td>
<td>219</td>
<td>140</td>
<td>88398</td>
</tr>
<tr>
<td></td>
<td>Hawally</td>
<td>LC2</td>
<td>91068</td>
<td>125</td>
<td>88</td>
<td>91281</td>
</tr>
<tr>
<td></td>
<td>Al Farwaniya</td>
<td>LC3</td>
<td>58834</td>
<td>242</td>
<td>166</td>
<td>59242</td>
</tr>
<tr>
<td></td>
<td>Al Ahmadi</td>
<td>LC4</td>
<td>48394</td>
<td>184</td>
<td>484</td>
<td>49062</td>
</tr>
<tr>
<td>2013</td>
<td>YE1</td>
<td>77334</td>
<td></td>
<td>199</td>
<td>246</td>
<td>77779</td>
</tr>
<tr>
<td>2014</td>
<td>YE2</td>
<td>86199</td>
<td></td>
<td>233</td>
<td>245</td>
<td>86677</td>
</tr>
<tr>
<td>2016</td>
<td>YE3</td>
<td>62347</td>
<td></td>
<td>165</td>
<td>242</td>
<td>62754</td>
</tr>
<tr>
<td>2017</td>
<td>YE4</td>
<td>60455</td>
<td></td>
<td>173</td>
<td>145</td>
<td>60773</td>
</tr>
</tbody>
</table>
3. Methodology

3.1. Applied model

In this study, a multinomial logit regression model was chosen to identify the significant variables and determine the correlation between predictors and the dependent variable. A multinomial logit regression was chosen as this model is flexible compared to ordered model, it can handle various types of variables that consist more than two categories. Moreover, the multicollinearity test was performed to determine if there is an issue when applying this model as it can be affected by multicollinearity between the independent variables. The test showed that there was no multicollinearity issue.
By following previous proceedings in applying the multinomial logit regression model (Ma, et al. 2014), the type of accident that represents the dependent variable and that includes crashes, run-over, and rollover accidents was assigned to be $y$ and crashes was assigned to be $y=1$, run-over accidents was assigned to be $y=2$, and rollover was assigned to be $y=3$ which is the reference level. Then $y=1$ and $y=2$ were compared to $y=3$. The $y=1$ and $y=2$ logit regression functions were as followed:

$$logit(p_1) = \ln \left( \frac{p(y = 1|x)}{p(y = 3|x)} \right) = \alpha_1 + \sum_{i=1}^{m} \beta_{1i} x_i \quad (1)$$

$$logit(p_2) = \ln \left( \frac{p(y = 2|x)}{p(y = 3|x)} \right) = \alpha_2 + \sum_{i=1}^{m} \beta_{2i} x_i \quad (2)$$

For both (1) and (2) equations, the value for the $i$th independent variable is $x_i$, $\alpha_1$ and $\alpha_2$ represented the intercept of the first and the second logit function respectively, the number variables in both equations (1) and (2) was represented by $m$. $\beta_{1i}$ and $\beta_{2i}$ represented the corresponding coefficient of the first and the second logit function respectively.

The condition probability of $k$th outcome category was:

$$p(y = k|x) = \frac{\exp(\alpha_k + \sum_{i=1}^{m} \beta_{ki} x_i)}{1 + \sum_{k=1}^{K-1} \exp(\alpha_k + \sum_{i=1}^{m} \beta_{ki} x_i)} \quad (3)$$

$\alpha_k$ is the intercept of the $k$th logit function, the corresponding coefficient of the $k$th logit function (3) is $\beta_{ki}$, and the number of outcome category is $K$. 
3.2. Results

Table 2 showed the maximum likelihood for the independent variables. As shown in table 2, both the location where the accident happened and the year when the accident occurred were significant variables as their p-value were less than 0.05. As a result, both predictors were included in the final multinomial logit regression model to determine the correlations.

Table 2. Maximum likelihood analysis

<table>
<thead>
<tr>
<th>Independent variables</th>
<th>Abbreviation</th>
<th>-2 Log Likelihood of Reduced Model</th>
<th>Chi-Square</th>
<th>Degree of freedom</th>
<th>P-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td>LC</td>
<td>1062.102</td>
<td>858.132</td>
<td>6</td>
<td>0.000</td>
</tr>
<tr>
<td>Year</td>
<td>YE</td>
<td>230.820</td>
<td>26.851</td>
<td>6</td>
<td>0.000</td>
</tr>
</tbody>
</table>

Table 3 showed the results of the applied the multinomial logit regression. As previously mentioned, rollover accidents were chosen to be the reference to investigate if there is a relationship of the location and time with the accident types.

Crashes were compared to rollovers in the first set of table 3, then run over was compared to rollovers. LC1, LC2, and LC3 that represented Kuwait City, Hawallly, and Al Farwaniya respectively were significant in the first and second set with positive coefficients and the reference level was Al Ahmadi governorate. This indicated that Al Ahmadi governorate had a higher probability to have rollover accidents compared to the three other governorates.

For the time when the accident occurred, YE1 and YE3 that represented 2013 and 2016 were significant in the first and second set with negative coefficients compared to the reference level which was 2017. This indicated that the year 2017 had a higher probability to have run-over compared to the two other years 2013 and 2016.
Table 3. Multinomial logit model for traffic accidents types

<table>
<thead>
<tr>
<th>Variable</th>
<th>Coefficient</th>
<th>Standard Error</th>
<th>Wald Chi-Square test</th>
<th>P-value</th>
<th>Odds ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Y=1 (crashes vs rollover)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Intercept</td>
<td>4.876</td>
<td>0.088</td>
<td>3056.081</td>
<td>0.000</td>
<td>6.311</td>
</tr>
<tr>
<td>LC1</td>
<td>1.842</td>
<td>0.096</td>
<td>366.914</td>
<td>0.000</td>
<td>10.389</td>
</tr>
<tr>
<td>LC2</td>
<td>2.341</td>
<td>0.116</td>
<td>406.955</td>
<td>0.000</td>
<td>10.639</td>
</tr>
<tr>
<td>LC3</td>
<td>1.271</td>
<td>0.090</td>
<td>198.550</td>
<td>0.000</td>
<td>3.563</td>
</tr>
<tr>
<td>YE1</td>
<td>-0.337</td>
<td>0.105</td>
<td>10.319</td>
<td>0.001</td>
<td>0.714</td>
</tr>
<tr>
<td>YE3</td>
<td>-0.501</td>
<td>0.105</td>
<td>22.591</td>
<td>0.000</td>
<td>0.606</td>
</tr>
<tr>
<td>Y=2 (run-over vs rollover)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Intercept</td>
<td>-0.635</td>
<td>0.132</td>
<td>23.054</td>
<td>0.000</td>
<td>4.139</td>
</tr>
<tr>
<td>LC1</td>
<td>1.421</td>
<td>0.139</td>
<td>104.947</td>
<td>0.000</td>
<td>3.755</td>
</tr>
<tr>
<td>LC2</td>
<td>1.323</td>
<td>0.164</td>
<td>65.148</td>
<td>0.000</td>
<td>3.863</td>
</tr>
<tr>
<td>LC3</td>
<td>1.352</td>
<td>0.133</td>
<td>103.390</td>
<td>0.000</td>
<td>3.863</td>
</tr>
<tr>
<td>YE1</td>
<td>-0.444</td>
<td>0.148</td>
<td>9.025</td>
<td>0.003</td>
<td>0.642</td>
</tr>
<tr>
<td>YE3</td>
<td>-0.578</td>
<td>0.151</td>
<td>14.591</td>
<td>0.000</td>
<td>0.561</td>
</tr>
</tbody>
</table>

4. Conclusions

This study was conducted to investigate the impacts of both location and time on three types of traffic accidents that occurred in Kuwait by utilizing official traffic accidents data that were collected from General Department of Traffic. Four governorates and years were included to distinguish the differences between them with respect to traffic accidents that resulted in 287,983 crashes, rollovers, and run-over accidents.

The statistics showed that the accidents decreased through the four years. In addition, governorate the had the higher rate of traffic accidents during the selected years was identified which was Hawally. For the rollover accidents, Al Ahmadi was recognized as the governorate that had the highest rollovers accidents compared to other governorates. Run-over accidents were significantly higher in Al Farwaniya governorate. Afterward, a multinomial logit regression was applied to identify the significant predictors and determine the correlation between independent variables and the dependent variable. The results showed that both the location and time were
significant variables related to traffic accidents types occurrence. The applied model confirmed that Al Ahmadi governorate had a higher probability of having rollover accidents compared to other governorates. Moreover, 2017 had higher odds of having run-over accidents compared to the other three selected years.

All in all, this study can be considered as a baseline for future studies that concern with traffic accidents in Kuwait due to the lack of similar studies. This study can help policy-makers, related institutions, police departments and ministry of interior in locating the area that is correlated with the three traffic accidents types. Additionally, this study can help in evaluating the newly established rules regarding mitigating and lessening traffic accidents through the selected time-period and locations in Kuwait.

References


Chen, Cong, Guohui Zhang, Zhen Qian , Rafiqul A. Tarefder , and Zong Tian . 2016. "Investigating driver injury severity patterns in rollover crashes using support vector machine models." Accident Analysis & Prevention 128-139.


KEYWORDS: Side impact crashes, NHTSA ratings, Injury severity, NASS CDS

EXTENDED ABSTRACT:
In 2015, side impact crashes ranked first in the cause of fatalities from motor vehicle crashes in the USA (National Highway Traffic Safety Administration, 2017). Side impact crashes are typically more devastating than frontal, and rear end crashes due to the limited space between the occupants and the colliding vehicle. This paper investigates the injury patterns in side impact crashes and compares the results across National Highway Traffic Safety Administration (NHTSA) vehicle safety ratings. In 2011, NHTSA made changes in the side impact test procedures and rating system: which allowed this study to make comparisons between old and new rating system. Further, driver injury severities in side impact crashes are compared with lower-rated and higher-rated vehicles. Odds ratios were computed and appropriate statistical tests were performed to study statistical significant differences. National Automotive Sampling System (NAAS) Crashworthiness Data System (CDS) was used for achieving the aim of this study. NASS CDS data from 2005 to 2014 which is publicly available on the NHTSA webpage is used for the analysis. Data for any given year are provided in 13 different SAS files that can be integrated using the unique fields such as primary sampling unit (PSU), case id, occupant number, and vehicle number. All the CDS crashes from 2005 to 2014 were aggregated to create one large database of a total 43,748 crashes. Several filters were further applied to this database to identify and select crashes that replicate NHTSA side crash test condition. Upper and lower extremities of the drivers are the body regions with most frequent injury outcomes in side impact crashes. Vehicles rated under old NHTSA rating criteria, the average Injury Severity Score (ISS) for younger drivers (aged ≤ 55 years) was 6.21 and 3.50 for lower- and higher-rated vehicles respectively: the difference was statistically significant at 95% confidence level. However, for older drivers the difference was not significant. Further, higher-rated vehicles in the new NHTSA rating system are significantly safer than the higher-rated vehicles in the old rating criteria. This paper establishes the fact that vehicles with higher star ratings (under experimental conditions) indeed offer increased occupant protection in the field conditions thus providing a point of reference to educate the drivers about the same.
KEYWORDS:
Transportation infrastructure deterioration
Pathology
Functional and structural evaluation
Sustainability.

ABSTRACT:
Over the past years, Ecuador experimented a significant development on its transportation infrastructure by implementing new roads and bridges. Which represents an upcoming responsibility on creating comprehensive maintenance and rehabilitation plans and management processes. On this path, the present work shows a complete methodology for functional and structural evaluation of transportation assets; specifically a bridge located in Cuenca- Ecuador, which creates a strong basis for procedures on infrastructure pathology and rehabilitation & reinforcement solutions.

The main objective of this work is to present maintenance, rehabilitation, and reinforcement solutions for a specific bridge based on the results obtained by semi, and non-destructive tests. Non-destructive methods embraces rebound hammer tests, concrete carbonation and remaining life examinations, ultrasound for cracks and density measurements, pachometer scans, and ground penetrating radar (GPR) measurements. Semi-destructive methods includes concrete core extractions on columns, beams, slabs, and arch elements.

With the obtained results on strength, cracks severity, reinforcement density and location, carbonation, etcetera; a subsequent structural modeling were performed in order to verify if the bridge complies with past and current construction codes. Finally, different solution alternatives were suggested, emphasizing in the more sustainable one for the analyzed structure, thus setting a precedent for future interventions on similar transportation assets.
Bridges are key transportation infrastructures needed for society in order to reach a sufficient and needed development. Economic growth is a result of an adequate transportation infrastructure, since a planned connectivity helps significantly saving time and money to the users. Socially speaking, relationships gets stronger for all sectors, not only commercially, but culturally as well. Safety is another important factor, and bridges most comply with all specifications, and be functionally and structurally adequate. Finally, on the other hand, is it also imperative to be environmentally friendly and mitigate any negative impact when constructing and maintaining the transportation infrastructure. With those aspects on mind, this work shows a significant part of the asset management process with a sustainable view on the evaluation phase, along with a correct presentation of solutions.

Usually, the most critical problem found on bridges is a deficient or bad design, that is a potential cause for collapse, but even with a good design, a significant percentage of problems are not due to a bad design but due to poor maintenance. Small deterioration processes become significant if not threatened on time, which is one of the most important aspects engineers should consider in order to manage infrastructure assets correctly. It is always better to reach a detailed design investing enough time at the beginning, than making potentially wrong decisions during the construction process (Helene & Pereira, 2007, p. 22). It is common to see old infrastructure with a lot of deterioration due to the environment, a natural aging process must be always controlled with an intelligent and adequate maintenance plan (Helene & Pereira, 2007). The bridge studied, and presented in this work “Puente sobre el río Tomebamba” is one of the most important connections in Cuenca City due to its significant traffic flow that has been incremented in the last decade because many city projects like the trolley. The study of its condition is therefore one of the most important aims for the City’s transportation management agency.

OBJECTIVES:

General Objective:

Perform a comprehensive structural and functional evaluation for the bridge: “Puente sobre el río Tomebamba” using semi and non-destructive methods, and to present solution alternatives that are sustainable.

Specific Objectives:

- Evaluate and diagnose all the different material son the bridge
- Identify all deterioration and its severity
- Evaluate the original structural design and verify its compliance of the current specifications.
- Propose solution alternatives, analyzing its advantages and disadvantages under LCA and LCCA criteria.
- Detail the solution alternative (or alternatives) that are sustainable to implement.

2 METHODOLOGY

The methodology is divided into three main parts: Forensic engineering, Data processing, and Solution Alternatives’ Analysis with a sustainable approach.

The following flow chart shows the applied methodology detailing all tests, tools and processes used in the study.

A. Forensic Engineering

A pathologic study was performed, including structural and functional evaluation. Among the tests and methods used on the structural evaluation are: geometric survey for all the bridge elements, steel detection, carbonations tests in order to find the remaining service life, cracks ultrasonic survey, concrete strength via rebound hammer testing and core braking; finally Ground Penetrating Radar (GPR) were used for delamination survey, layers health, etcetera.

For the functional evaluation, a Pavement Condition Index (PCI) survey were performed.

B. Data processing

Knowing the steel position and quantity, and the geometric survey, a precise database was created in order to start the building of the structural modeling; also, precise compressive strength of concrete were found for each element, by correlations developed between the rebound hammer tests and the compressive tests results from braking the extracted cores;

Remaining life due to carbonation was also estimated; cracks’ position, depth, and total relative volume were found with ultrasonic tests. Finally, structural modeling and validation was performed using SAP 2000 V.18; specification compliance for previous (for the year of the bridge construction), and current normative was also performed.

C. Sustainability

LCA, identifying all phases in the bridge life cycle, and evaluating its environment impact was performed, the phases were:
- Objective definition and scope
- Inventory analysis of the life cycle.
- Impact assessment of the life cycle.
- Interpretation.

After this a LCCA was also performed, evaluating the proposed solution alternatives in order to define the most effective treatment, it is important to state that user costs were also considered, not only agency costs. The following are the steps in the LCCA process:
- Estimate costs
- Compute LCCA.
- Analyze results
- Chose the best alternative.

3 RESULTS

3.1. Forensic Engineering
Pathological and Structural Evaluation

The following Figures, Fig. 2 y Fig. 3 shows the plant and front view of the studied bridge according to the geometrical survey.

For transversal and longitudinal steel inspection, the British Standard BS1881 were followed, quantity, position, depth were defined for all elements. For diameters verification and fulfillment of the measurements, a semi-destructive inspection using hammers, chisels, and calibrator were used.

Carbonation tests with phenolphthalein at 1% in alcohol, following the American Society for Testing and Materials (ASTM) D1293 were also performed over the main elements, carbonation depths and remaining service life due to CO₂ attack was calculated.

Rebound hammer tests were performed following the ASTM C-805, abrasive rock for cleaning, and at least 12 hammer rebounds (separated more than 3 centimeters) were performed over each element.
Cracks survey were done using the ultrasonic test following the ASTM C597 standard, the measurements were made following the “superficial”, and/or “indirect” method. Initially the locations of the higher crack widths/openings were defined in order to measure the higher depths. A schematic view for the method is presented in the following figure, Fig. 4.

![Fig. 4. Separation of transducers for the crack depth measurements](image)

GPR measurements were performed over the entire superstructure, following the ASTM D6432, in order to know the subsurface characteristics of the bridge, such as steel locations, concrete homogeneity, delamination, corrosion potentials due to water presence, et cetera.

Core extractions were also performed (as semi-destructive tests) following the ASTM C42 standard, this tests were performed in the most critical locations, meaning locations with the lower sclerometric indices, and covering all types of elements, 6 cores were extracted on beams, columns, arch and superstructure slabs.

**Functional Evaluation**

PCI survey over all lanes was performed, following the ASTM D6433 standard; aiming to know the current state of the asphalt layer, different deterioration types and severities were found, following the mentioned standard and different deterioration definition manuals (Vásquez, 2002), border cracks, joint reflection cracks, patches, delamination and spalling were the main found deteriorations.

### 3.2. Data Processing

- **Steel detection**

The following figures shows the type, location, and quantity for the steel reinforcement on different elements conforming the bridge structure.

![Fig. 5. Steel reinforcement in columns](image)
• Carbonation test

Table 1 shows as an example the results for the columns located at the right side, same tests were performed for the rest of the elements; generally, carbonation tests do not show significant CO₂ advances.

<table>
<thead>
<tr>
<th>Element</th>
<th>depth(cm)</th>
<th>Covering (cm)</th>
<th>remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>0,9</td>
<td>6</td>
<td>Carbonation reaches 15% of the concrete covering</td>
</tr>
<tr>
<td>C2</td>
<td>1</td>
<td>6</td>
<td>Carbonation reaches 17% of the concrete covering</td>
</tr>
<tr>
<td>C3</td>
<td>0,6</td>
<td>6</td>
<td>Carbonation reaches 10% of the concrete covering</td>
</tr>
<tr>
<td>C4</td>
<td>3</td>
<td>6</td>
<td>Carbonation reaches 50% of the concrete covering</td>
</tr>
<tr>
<td>C5</td>
<td>0,1</td>
<td>6</td>
<td>Carbonation is irrelevant</td>
</tr>
<tr>
<td>C6</td>
<td>1,5</td>
<td>5</td>
<td>Carbonation reaches 30% of the concrete covering</td>
</tr>
</tbody>
</table>
The remaining life were calculated using K. Tuutti modeling (1982), detailed as follows:

\[
\begin{align*}
    d &= k \sqrt{t} \\
    t &= \left(\frac{4}{k}\right)^2
\end{align*}
\]

Where:
- \(d\) = carbonation depth (mm)
- \(k\) = carbonation coefficient
- \(t\) = time (years).

The remaining service life were calculated only for the most critical elements, assuming a service life of 50 years (normally used for bridges).

Table 2 summarizes the results, Fig.9 schematize them for beams 2.3, 4.2 y 6.1

<table>
<thead>
<tr>
<th>Element</th>
<th>Depth (cm)</th>
<th>Carbonation rate (k)</th>
<th>Concrete Covering (mm)</th>
<th>time (years)</th>
<th>Remaining life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V2.3</td>
<td>4,9</td>
<td>7,30</td>
<td>20</td>
<td>7,50</td>
<td>-37,50</td>
</tr>
<tr>
<td>V3.3</td>
<td>1,3</td>
<td>1,94</td>
<td>10</td>
<td>26,63</td>
<td>-18,37</td>
</tr>
<tr>
<td>V4.3</td>
<td>1,7</td>
<td>2,53</td>
<td>10</td>
<td>15,57</td>
<td>-29,43</td>
</tr>
<tr>
<td>V6.1</td>
<td>2</td>
<td>2,98</td>
<td>20</td>
<td>45,00</td>
<td>0,00</td>
</tr>
<tr>
<td>C4</td>
<td>3</td>
<td>4,47</td>
<td>60</td>
<td>180,00</td>
<td>135,00</td>
</tr>
<tr>
<td>T8</td>
<td>0,2</td>
<td>0,30</td>
<td>20</td>
<td>4500,00 *</td>
<td>4455,00</td>
</tr>
<tr>
<td>A15</td>
<td>1,2</td>
<td>1,79</td>
<td>40</td>
<td>500,00*</td>
<td>455,00</td>
</tr>
</tbody>
</table>

*Theoretically the remaining service life for the arch is significantly big; but, this is not the real case since missing concrete covering was observed, meaning no remaining service life due to direct environmental attack.

Experimental data helps us to classify concrete quality as follows (Rondón, 2005)
Table 3. Concrete quality as a function of $K$

<table>
<thead>
<tr>
<th>$K$</th>
<th>Concrete characteristics</th>
<th>Concrete quality</th>
</tr>
</thead>
<tbody>
<tr>
<td>$2&lt;k&lt;6$</td>
<td>High compactness, cement content &gt; 350Kg/m³</td>
<td>Good</td>
</tr>
<tr>
<td>$6&lt;k&lt;9$</td>
<td>Average compactness, cement content $\geq 250 \leq 350$ Kg/m³</td>
<td>Average</td>
</tr>
<tr>
<td>$K&gt;9$</td>
<td>Porous, cement content &lt; 250Kg/m³ &amp; high w/c ratio</td>
<td>Bad</td>
</tr>
</tbody>
</table>

As shown in

Table 2 and Table 3 the element V2.3 has an “average” quality, differing from all the rest that shows a “good” quality.

- **Rebound hammer test**
  Corrections due to carbonation were performed over the sclerometric results (Fernández, 2013).

Table 4 shows the results for the Arch, notice that the table shoes only seven rebounds, but at least 12 rebounds were performed for each test, same for the rest of the elements.

Table 4. Sclerometric test on the arch (extract)

<table>
<thead>
<tr>
<th>Element</th>
<th>Number of Rebounds ($N$)</th>
<th>Ave. N</th>
<th>Correction factor</th>
<th>Corrected N</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>54 55 57 56 57 49 53</td>
<td>55</td>
<td>1</td>
<td>55.00</td>
</tr>
<tr>
<td>A3</td>
<td>61 53 54 60 57 53 57</td>
<td>57</td>
<td>1</td>
<td>57.00</td>
</tr>
<tr>
<td>A4</td>
<td>39 37 48 40 42 39 40,4</td>
<td>40,4</td>
<td>1</td>
<td>40,40</td>
</tr>
<tr>
<td>A6</td>
<td>52 51 52 48 48 55 52</td>
<td>52,3</td>
<td>0,97</td>
<td>50,47</td>
</tr>
<tr>
<td>A10</td>
<td>44 50 48 45 53 50 49</td>
<td>48,6</td>
<td>0,97</td>
<td>46,90</td>
</tr>
<tr>
<td>A12</td>
<td>54 58 60 55 57 54 56</td>
<td>57,1</td>
<td>0,97</td>
<td>55,10</td>
</tr>
<tr>
<td>A13</td>
<td>53 55 49 51 51 54 47</td>
<td>51,9</td>
<td>0,97</td>
<td>50,08</td>
</tr>
<tr>
<td>A15</td>
<td>47 54 52 50 48 48 50</td>
<td>50,6</td>
<td>0,93</td>
<td>47,06</td>
</tr>
</tbody>
</table>

Note: for all elements the equipment was horizontal during the test

- **Ultrasound test**
  Table 5 shows the most relevant results for cracks’ depth and width, the used distance between transducers (b) was 5 cm for all cases. The most relevant cracks are highlighted according to the American Concrete Institute (ACI) 224R.

Table 5. Ultrasound test

<table>
<thead>
<tr>
<th>Element</th>
<th>t1 (μs)</th>
<th>t2 (μs)</th>
<th>depth “d” (cm)</th>
<th>with (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Abutment MD</td>
<td>∞</td>
<td></td>
<td>10 through crack</td>
<td></td>
</tr>
<tr>
<td>Abutment MI</td>
<td>∞</td>
<td></td>
<td>3 through crack</td>
<td></td>
</tr>
<tr>
<td>T5</td>
<td>250</td>
<td>528</td>
<td>0,5 through crack</td>
<td></td>
</tr>
<tr>
<td>T6</td>
<td>104,7</td>
<td>120</td>
<td>1,1</td>
<td></td>
</tr>
<tr>
<td>V3.3</td>
<td>97,8</td>
<td>190,2</td>
<td>1,4</td>
<td>0,1</td>
</tr>
<tr>
<td>V3.3</td>
<td>177</td>
<td>351</td>
<td>0,8</td>
<td>0,2</td>
</tr>
</tbody>
</table>
Table 6 shows ultrasonic wave speed (velocity), and the correspondent concrete quality is shown in Table 7.

**Table 6. Ultrasonic pulse velocity for all elements**

<table>
<thead>
<tr>
<th>Element</th>
<th>t1 (μs)</th>
<th>distance &quot;d&quot; (cm)</th>
<th>velocity &quot;v&quot; (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>91</td>
<td>10</td>
<td>1100</td>
</tr>
<tr>
<td>C2</td>
<td>33,6</td>
<td>10</td>
<td>2890</td>
</tr>
<tr>
<td>C7</td>
<td>101</td>
<td>30</td>
<td>2970</td>
</tr>
<tr>
<td>C9</td>
<td>94,7</td>
<td>30</td>
<td>3170</td>
</tr>
<tr>
<td>C15</td>
<td>119</td>
<td>30</td>
<td>2520</td>
</tr>
<tr>
<td>V2.3</td>
<td>484</td>
<td>30</td>
<td>620</td>
</tr>
<tr>
<td>T9</td>
<td>106</td>
<td>10</td>
<td>940</td>
</tr>
<tr>
<td>T15</td>
<td>314</td>
<td>30</td>
<td>960</td>
</tr>
<tr>
<td>A4</td>
<td>64</td>
<td>10</td>
<td>1570</td>
</tr>
<tr>
<td>A10</td>
<td>151</td>
<td>20</td>
<td>1330</td>
</tr>
<tr>
<td>E4</td>
<td>478</td>
<td>30</td>
<td>630</td>
</tr>
<tr>
<td>E7</td>
<td>177</td>
<td>30</td>
<td>1690</td>
</tr>
</tbody>
</table>

**Table 7. Concrete quality due to ultrasonic speed**

<table>
<thead>
<tr>
<th>wave speed m/s</th>
<th>Quality of the concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;4570</td>
<td>Excellent</td>
</tr>
<tr>
<td>3050-4570</td>
<td>Good</td>
</tr>
<tr>
<td>3050-3650</td>
<td>Regular</td>
</tr>
</tbody>
</table>
Concrete quality for the different elements varies from “regular” to “very poor”.

- **Ground Penetrating Radar test (GPR)**
  
  Fig.10 shows column C2 reinforcement, it is noticeable that Steel spacing’s are approximately equal, and consistent with other testing (Steel survey using the pachometer), around 12 cm.

![Fig. 10. GPR results for column C2](image)

Reinforcement discontinuities on the main slab are visible as shown in previous figure and Fig.11, this correlates perfectly with the places where corrosion and humidity were found, it is also observed that the asphalt layer thickness is around 5 cm.

![Fig. 11. GPR results for the main slab (in 2D)](image)

Fig.12 shows a 3D image of the main slab, it is noticeable that on the two right lanes the presence of corrosion, as stated before.

![Fig. 12. GPR results for the main slab (3D)](image)

Fig.13 shows beam V2.2 with no continuous spacing on the steel abutments; this recalls a lack of quality control during construction.
- Core extractions

Table 8 shows a summary of the concrete compression strength results.

<table>
<thead>
<tr>
<th>Element</th>
<th>weight (kg)</th>
<th>f'c (Kg/cm²)</th>
<th>(h/d) ratio</th>
<th>Correction factor</th>
<th>Corrected f'c (Kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column 1</td>
<td>0,91</td>
<td>161,3</td>
<td>1,56</td>
<td>0,951</td>
<td>153,39</td>
</tr>
<tr>
<td>Beam 2.3</td>
<td>3,42</td>
<td>130,2</td>
<td>2,01</td>
<td>1</td>
<td>130,2</td>
</tr>
<tr>
<td>Beam 6.1</td>
<td>1,84</td>
<td>174,3</td>
<td>1,14</td>
<td>0,904</td>
<td>157,6</td>
</tr>
<tr>
<td>Stab 6</td>
<td>1,78</td>
<td>223,4</td>
<td>1,12</td>
<td>0,899</td>
<td>200,8</td>
</tr>
<tr>
<td>Arch 1</td>
<td>1</td>
<td>266,36</td>
<td>1,72</td>
<td>0,978</td>
<td>260,5</td>
</tr>
<tr>
<td>Arch 10</td>
<td>1,1</td>
<td>225,8</td>
<td>1,75</td>
<td>0,98</td>
<td>221,3</td>
</tr>
</tbody>
</table>

With these results, and the average sclerometric number for the correspondent element, the following correlations were Fig.14.

Equation (3) is defined, with a R2 = 0.905, (confidence level of 90.5%).

\[
y = 0.1687x² - 9.092x + 251,28
\]

(3)

With equation (3), it was possible to determine the compressive strength for all elements indirectly, but with a very good confidence level.

Table 9 shows the average compressive strengths for all elements of the bridge. The average compressive strength for beams and columns do not comply with the minimum (210 Kg/cm²) required for the ACI 318-14. On the other hand, the strength for diaphragms and arches do comply with the minimum. The average values for the main slab are shown in Table 9, it is noticeable that the value is lower than the minimum (280kg/cm²) required for the AASHTO LRFD-04.
Before defining different rehabilitation treatments for the bridge in following chapters, it is appropriate to establish minimum requirements for specific potential treatments as follows:

In order to apply carbon fiber reinforcements (fiber reinforcement polymers) it is required for the element that is going to be treated to have a minimum tensile strength of 1.4 MPa (Helene & Pereira, 2007). Table 10 shows this strength for the most critical element, showing that even this “weak” element does comply with the specification.

Table 10. Specified minimum tensile strength check

<table>
<thead>
<tr>
<th>Element</th>
<th>Tensile strength: $f_{ct}$ (MPa)</th>
<th>$f'_c$ (kg/cm$^2$)</th>
<th>$f_{ct}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V2.3</td>
<td>$0.3 \sqrt{f'_c}$</td>
<td>131.05</td>
<td>1.6</td>
</tr>
</tbody>
</table>

**Structural Analysis**

**Specification Compliance**

- **Columns**
  Knowing that: minimum requirement for longitudinal steel quantity ($\rho$) in columns is $0.01 \leq \rho \leq 0.03$, minimum spacing between bars is 4-cm. minimum diameter is 16mm, minimum covering is 4 cm; it was found that: the steel in columns does comply with: quantity, spacing and diameter, but four of the sixteen columns does not comply with covering.
  For traversal Steel: minimum diameter is 10 mm, maximum spacing between abutments: 10 cm. It was found that none of these requirements were met, in any column.

- **Beams**
  Requirements were met for longitudinal steel quantities: $0.003 \leq \rho \leq 0.025$, none of the covering met the criteria ($> 4$cm). Minimum transversal steel diameters were met ($> 10$mm).
  None of the maximum spacing between abutments were met, required maximum spacing is 8.5cm, and it was found that average spacing were 25-cm. notice that this requirement presents a significant flaw.

- **Slab**
  Requirements for upper and lower longitudinal steel are met, as well as spacing, but with 2 cm of concrete covering, this specific requirement is not met.
  In the case of transversal steel, all requirements were met.

- **Arch**
  Requirements are met for longitudinal and transversal steel ($>60$cm$^2$), diameter ($>10$mm), and covering ($>4$ cm.), but it is not the case for maximum spacing.

**Sizing**

For sizing, the following standards: AASHTO LRFD, ACI 318S-14 and the Ecuadorian Construction Standard: NEC-SE-HM-14; were checked and presented below:
According to NEC-SE-HM-14, chapter 4.3.1 specifies a minimum width of 30cm for columns, the bridge’s columns comply with this standard (bridge’s columns are 30x30cm).

As shown in Fig. 2, transversal beams are configured in sets of three, with 35 cm height and 30 cm width, this configuration complies with ACI 318S-14., ACI 318S-14, and NEC-SE-HM-14.
The four 3-meter lanes also complies with AASHTO LRFD, table 2.5.2.6.3-1
According to the standards, minimum asphalt layer height is:

\[
\frac{S + 3000}{30} \geq 165mm
\]

Where: S is the effective length of the bridge.
The bridge complies with this minimum; having 22cm. also complies with AASHTO LRFD- article: 13.7.3.1.2.

According to “Puentes y obras de arte” (Pastor, 2000), the minimum arch width is 0.015(9.15+L) = 49.7 cm
Where L is the arch length, The bridge’s arch of 50 cm complies with this.

Structural Modeling
SAP2000 V18 was used for this phase, along with AutoCAD Civil 3D.
Loads include bridge weight and railings (DC), pavement (DW), horizontal force (HH), vehicles (LL), people (PL).
Vehicle loads are modeled as and HL-93 (3-axle tandem truck).

Load combinations were developed according to AASHTO LRFD-04 tables: 3.4.1-1 and 3.4.1-2; being the most critical combination as follows:
COMB 7 = 1.25 DC + 1.5 DW + 1.35 EH + 2.33 (LL) + 1.75 (PL)

The Structural modeling can be seen on Fig. 15 below.

![Fig. 15. Bridge structural modeling studied on SAP 2000](image)

Fig. 16 below shows the vertical deformation diagram for the slab including sidewalks according to COMB 7, Maximum deformations are shown in the middle of the slab, Maximum deflection (19.6mm) complies with the standards (L/1000= 28.37 mm).
Table 11 compares required steel (to hold the most critical combination of loads) versus the actual steel found in the bridge.

Table 11. Required vs. used steel

<table>
<thead>
<tr>
<th>Element</th>
<th>Unit</th>
<th>Required steel</th>
<th>Used steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column</td>
<td>Longitudinal steel</td>
<td>cm²</td>
<td>9</td>
</tr>
<tr>
<td>Longitudinal beam</td>
<td>lower</td>
<td>cm²</td>
<td>2.38</td>
</tr>
<tr>
<td></td>
<td>upper</td>
<td>cm²</td>
<td>5.86</td>
</tr>
<tr>
<td>Transversal</td>
<td>Av</td>
<td>cm²</td>
<td>1.15</td>
</tr>
<tr>
<td>Slab</td>
<td>Longitudinal</td>
<td>cm²/m</td>
<td>12.01</td>
</tr>
<tr>
<td></td>
<td>lower</td>
<td>cm²/m</td>
<td>14.78</td>
</tr>
<tr>
<td></td>
<td>upper</td>
<td>cm²/m</td>
<td>3.92</td>
</tr>
<tr>
<td></td>
<td>Transversal</td>
<td>cm²/m</td>
<td>8.91</td>
</tr>
<tr>
<td></td>
<td>lower</td>
<td>cm²/m</td>
<td>18.27</td>
</tr>
<tr>
<td></td>
<td>upper</td>
<td>cm²/m</td>
<td>8.78</td>
</tr>
<tr>
<td>Arch</td>
<td>Longitudinal</td>
<td>cm²/m</td>
<td>5.12</td>
</tr>
<tr>
<td></td>
<td>lower</td>
<td>cm²/m</td>
<td>3.04</td>
</tr>
</tbody>
</table>

3.3. Alternatives’ Analysis

Taking into account that the main pathologies found on the bridge are:
- Generalised corrosion
- Superficial holes and cracks over the slab, arch, and beams
- Efflorescence on beams, slab, and abutments
- Delamination on the concrete slab and arch
- Tension cracks on beams
- Aggregates’ breakdown on the superficial layer

The proposed treatment alternatives are:
1) Cracks’ treatment with epoxy resin’s injection
2) Cathodic protection
3) Modified mortar for repairing
4) Covering Paint
5) Joints’ waterproofing
6) Overlay asphalt treatment (simple & rubberized)
7) Grouting, among the main treatments
3.3.1. LCA

- **Objective definition & scope**
  The project frames maintenance and rehabilitation strategies for the specific main distresses and pathologies found in the bridge, by using a broad spectrum of products in order to stop deterioration and recover functional and structural adequacy.

  The project does not includes, within the proposed strategies, structural reinforcement of the bridge (in an objective and quantitative way), although recommendations for reinforcement using FRP materials are presented below.

  Alternatives were chosen taking into account minimum use of energy and environmental pollution.

- **Inventory**
  Database were created and analyzed using different sources, being the main ones BEDEC database (ITEC, 2017), Sika corporation, Sika Ecuatoriana («Sika Ecuatoriana», 2017)

- **Life cycle assessment and evaluation**
  Table 12 shows a comparative analysis for the proposed alternatives with focus and the environmental impact for each treatment

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Kg CO2/kg</td>
<td>Energy kWh/kg</td>
<td>Kg CO2/kg</td>
</tr>
<tr>
<td>Micro cement injection</td>
<td>1</td>
<td>1,262</td>
<td>Epoxy resin injection</td>
</tr>
<tr>
<td>Epoxy resin</td>
<td>13,91</td>
<td>24,81</td>
<td>Emulsion resins</td>
</tr>
<tr>
<td>Grout</td>
<td>0,4</td>
<td>0,61</td>
<td>Polymeric mortar</td>
</tr>
<tr>
<td>Polyurethane paint</td>
<td>5,87</td>
<td>33,06</td>
<td>Acrylic resin paint</td>
</tr>
<tr>
<td>Zinc + hydrogel</td>
<td>1,72</td>
<td>N.A</td>
<td>Cathodic protection</td>
</tr>
<tr>
<td>Silicone</td>
<td>7,39</td>
<td>45,43</td>
<td>Polyurethane resin base</td>
</tr>
<tr>
<td>Rubberized superficial treatment</td>
<td>11,81</td>
<td>24,79</td>
<td>Simple superficial treatment</td>
</tr>
</tbody>
</table>

Notes: ND= Not available
Highlighted values show the lower environmental impact

According to scientific literature, in Ecuador, the CO2 emission factor is 0.5076 Kg CO2/kWh (Ministerio del ambiente, 2013), this information (among others) were included in the decision making process in order to determine which alternative is more environmentally friendly when comparing specific treatments for the same benefit.

- **Performance Interpretation**
  As mentioned before Table 12 summarizes the results from an environmental view, and this information were used to decide the best treatment, for example: micro cement injection is more environmentally friendly than epoxy resin injection, grout is better than polymeric mortar, rubberized asphalt overlay treatment is better than simple superficial treatment, etcetera.

3.3.2. LCCA

Table 13 shows the costs for different materials, equipment, and man-work time, for each treatment presented on each alternative.

<table>
<thead>
<tr>
<th>Alternative 1</th>
<th>Life cycle total cost</th>
<th>Alternative 2</th>
<th>Life cycle total cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Micro cement injection</td>
<td>7,56 $/m</td>
<td>Epoxy resin injection</td>
<td>9,98 $/m</td>
</tr>
<tr>
<td>Epoxy resin</td>
<td>9,26 $/m2</td>
<td>Emulsion resins</td>
<td>5,9 $/m2</td>
</tr>
<tr>
<td>Grout</td>
<td>111,73 $/m2</td>
<td>Polymeric mortar</td>
<td>51,55 $/m2</td>
</tr>
</tbody>
</table>
It is important to mention too that the costs: relative to environmental pollution (the cost of each CO2 Tn. released into the environment when implementing the solution alternatives) and energy, used were also considered in the analysis.

Cost of a Ton of CO2 released equals 222USD i.ambiente, 2015)
Cost of energy in Ecuador equals 9.33 Cents/kWh (El comercio, 2016).

Table 14 and Table 15 show this information for each alternative.

Table 14. Environmental impact cost for Alternative 1

<table>
<thead>
<tr>
<th>Alternative 1</th>
<th>Environmental impact</th>
<th>Cost due to environmental impact</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Kg CO2/kg</td>
<td>Energy kWh/kg</td>
</tr>
<tr>
<td>Micro cement injection</td>
<td>1</td>
<td>1,262</td>
</tr>
<tr>
<td>Epoxy resin</td>
<td>13,91</td>
<td>24,81</td>
</tr>
<tr>
<td>Grout</td>
<td>0,4</td>
<td>0,61</td>
</tr>
<tr>
<td>Polyurethane paint</td>
<td>5,87</td>
<td>33,06</td>
</tr>
<tr>
<td>Zinc + hydrogel</td>
<td>1,72</td>
<td>N.A</td>
</tr>
<tr>
<td>Silicone</td>
<td>7,39</td>
<td>45,43</td>
</tr>
<tr>
<td>Rubberized superficial treatment</td>
<td>11,81</td>
<td>24,79</td>
</tr>
</tbody>
</table>

Note: N.A= Not available

Table 15. Environmental impact cost for Alternative 2

<table>
<thead>
<tr>
<th>Alternative 2</th>
<th>Environmental impact</th>
<th>Cost due to environmental impact</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Kg CO2/kg</td>
<td>Kg CO2/kg</td>
</tr>
<tr>
<td>Epoxy resin injection</td>
<td>13,21</td>
<td>24,41</td>
</tr>
<tr>
<td>Emulsion</td>
<td>9,61</td>
<td>18,11</td>
</tr>
<tr>
<td>Polymeric mortar</td>
<td>13,21</td>
<td>24,81</td>
</tr>
<tr>
<td>Acrylic resin paint</td>
<td>14,76</td>
<td>27,32</td>
</tr>
<tr>
<td>Cathodic protection</td>
<td>1,72</td>
<td>N.A</td>
</tr>
<tr>
<td>Polyurethane resin base</td>
<td>4,16</td>
<td>23,89</td>
</tr>
<tr>
<td>Simple superficial treatment</td>
<td>12,69</td>
<td>25,18</td>
</tr>
</tbody>
</table>

Note: N.A= Not available

Fig.17 and Fig.18 present the total LCA costs for each intervention, including pollution and energy costs.
Since it was possible to convert environmental impact into costs, an objective way to choose the best alternative were implemented. Therefore, alternative one were chosen as the best alternative (being the most environmentally friendly), with exceptions in particular treatments like cathodic protection, were sacrifice anodes are better than zing + hydrogel treatment.

Reinforcement recommendations
For low compressive strength elements (under the minimum recommended values according to the mentioned standards), reinforcement using fiber reinforcement polymers FRP materials is recommended.

The elements that must be reinforced are all the columns (except C2, C3, C5, and C11), all the beams (except V7)

4 DISCUSSION
- Flaws and irregularities in steel spacing were found in most elements like columns or beams, it is believed that a deficient quality control during construction is the main cause for this pathology
- Carbonation was expected in a more aggressive stage, but it was not the case, even though corrosion was significant around the main structural elements, it is believed therefore than this pathology is mainly caused by the synergy of humidity, porosity and holes in the concrete.
- According to the ultrasound test, concrete quality is defined into the “regular to very poor” range, it is believed that the high porosity observed influenced in the wave velocities during the testing. These results verify cores samples testing and sclerometric testing in almost all cases.
- It was not possible to extract core samples for the abutments due to physical difficulties, although, the sclerometric measurements in these elements were calibrated with other compressive strength results for the other elements obtaining a satisfactory precision in the predictions.

5 CONCLUSIONS
- It was possible to propose a sustainable solution alternative with a number of treatments that will help the bridge to renew its functionality and its structural adequacy.
- For the pathological analysis, steel detection was critical in order to correctly define locations, spacing, and quantities for all elements, results that were used for deficiencies definitions, standard compliance analysis, and structural modeling, although it is important to notice that this wide-ranging analysis was complemented and/or verified with GPR testing.
- Humidity and/or carbonation has reached the steel reinforcement severely due to the lack or absence of covering, that is the case for beams V2, V3, V4 y V6, were active corrosion was already found. There is no remaining life due to carbonation for all elements with exceptions to columns.
- It was concluded that it is imperative to build particular curves for rebound hammer indices, as the ones developed in this work, correlating sclerometric indices with real compressive strength results using cores; since the ones presented by the manufacturer overestimate the compressive strength values (over 400kg/cm² compared with real values below 300 kg/cm²).
Recalling the standardized compressive strength limits for columns and beams of 210 kg/cm² and 280 kg/cm² the slab; the results allows concluding that most of these elements do not comply with the standards.

According to ACI 224R, most of the cracks found in the bridge elements are significant, ultrasound testing also showed that, some of these cracks go all the way through the elements. The most important cracks are located in the upper beams, meaning that these cracks are tension/flexion cracks; it is imperative therefore that this tie of cracks must be repaired as soon as possible.

- the structural modeling shows that, even though the amount of transversal steel is just enough to support shear stress and moment, it is not enough to comply with the standards

Covering for: columns C12 & C16, all beams, and main slab; does not comply with the minimum. Key reason for accelerated deterioration and main cause for the corrosion found all over the reinforcement steel.

- Finally, the sustainable approach taken for the alternatives’ analysis allowed establishing the best solution treatments, with the lower CO₂ emissions and the lower energy consumption.

REFERENCES


ABSTRACT:
To minimize the impacts of conventional construction of new bridges or the replacement/rehabilitation of existing bridges, the transportation authorities have focused on implementing innovative Accelerated Bridge Construction (ABC) that attempts to shift most of the construction work off-site. The transportation agencies have developed higher-level decision-making tools to decide on the suitability of ABC for a specific bridge project. Despite this effort, currently, there are not any business case processes for ABC to help the stakeholders decide on whether or not to choose ABC in the early decision-making phase. So, to fulfill this requirement, the research has been carried out to develop a simple business case process flowchart for ABC. This process can be used for any bridge project during the early initial planning phase to assist the decision makers to decide between ABC and conventional bridge construction. This can benefit the industry, as it will result in an optimum value of modularization in this sector.
Business Case Process for Accelerated Bridge Construction

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1 INTRODUCTION

The conventional method of bridge construction requires a longer construction duration, which mainly causes traffic delay resulting in the wastage of time and money of the road users, and the disturbed traffic affects the local economy. Besides, there are many other disadvantages of the conventional bridge construction method that has forced the transportation authorities to shift towards Accelerated Bridge Construction (ABC). ABC is a bridge construction method that shifts most of the construction work away from the actual job site, reducing the onsite construction duration of bridges. There are two ways ABC method can be implemented (HNTB 2014). First, as suitable for the specific project, off-site prefabrication of bridge elements for both substructure and superstructure components of the bridge or prefabrication of any components. These fabricated components require transportation from the fabrication yard to the site followed by the use of heavy lifting equipment for final placement. Second, constructing the whole bridge beside/nearby the bridge location and then using the movement technology like sliding, rolling or skidding, and self-propelled modular transporters (SPMTs) to align it in the final position. As the majority of construction activities take place off-site, it minimizes the actual site construction work resulting in minimum traffic impact and improved safety of the workers and the traveling public.

The state Department of Transportation (DOTs), Transportation Research Board (TRB), the American Association of State Highway and Transportation Officials (AASHTO), Federal Highway Administration (FHWA), and academia and industry partners have collaborated together for the accelerated implementation and significant advancement of ABC in the recent years with lots of research projects on ABC (Ralls 2014). Though ABC has been successfully implemented in many bridge projects, it is still lagging behind the industrial sector modularization. For the industrial projects, the Construction Industry Institute (CII) has developed a modularization business case process (O’Brien et al. 2015) and an analysis tool as well (Choi and O’Connor 2015), which help to identify the optimum number of work hours that can be shifted away from the actual job site with specific value of savings. However, a similar type of business case process applicable to the infrastructure sector has not been developed yet. Therefore, there is a need of a business case process to help the decision makers easily determine the applicability of ABC on specific bridge projects early in the initial phase of planning, which will result in an optimal value of modularization in this sector. The implementation of accelerated construction will benefit the local people and business/industry, the road users/traveling public, state DOTs and contractors, and the environment.

2 LITERATURE REVIEW

2.1 DECISION-MAKING TOOLS

El-Diraby and O’Connor (2001) proposed a model for evaluation of the effectiveness of various bridge construction plans during the design phase based on inputs from actual construction sites in Texas, and industry experts in design and construction. The model involves calculation of a final score for each construction plan. The model considered accessibility, safety, carrying capacity, and schedule and budget performances as the major factors, and an additional 22 sub-factors. Based on the opinion of industry experts, these five factors are given relative weight in decision making. Further, each subfactor is given a score of 1-10, 10 being the best score. The score of a particular factor is calculated as the weighted sum of all its sub-factors. Then the final score for each identified construction plan is computed as the summation of multiple of weight and score of each sub-factor. The best construction plan is the one with the highest score. This model is used to identify the optimized construction plan to minimize the effect of construction on the surrounding area and improve the overall project performance.

The Federal Highway Administration (FHWA 2005) has developed a framework to help the decision makers to decide on the applicability of prefabricated bridge construction on a specific bridge project/location based on
economy and effectiveness. The framework is a decision-making tool that comprises three sections for high-level decision-making. The first one is a flowchart, which is based on cost comparison for decision-making. The second one comprises a more detailed Yes/Maybe/No matrix questionnaire where a maximum number of “Yes” answers indicate preference of prefabricated bridge construction. These two sections can be carried out separately or in conjunction. The third section includes a thorough discussion of various factors on the use of prefabrication. The discussion section is considered for in-depth assessment of various factors corresponding to the responses on matrix questionnaires related to safety and other site issues, environmental issues, onsite construction, standardization, and costs related to traffic maintenance and operations, and service life of the bridge.

Salem and Miller (2006) and Salem et al. (2013) proposed a decision-making process using AHP to determine the most cost-effective method suitable for bridge construction/rehabilitation. This process uses pairwise comparison of various factors such as costs, impact on traffic flow, safety, impact on society, impact on local economy, and environmental concerns. Each of these factors is responsible for successful completion of the project along with their corresponding sub-factors to determine the relative importance value for each factor and sub-factor in terms of numerical/verbal scale for all the identified bridge construction alternatives. A consistency ratio is calculated to determine consistency among the pairwise comparisons. Besides cost evaluation, this process incorporates evaluation of other non-technical factors as well that affect the decision-making process. Thus, applying this process, the decision makers can justify the selection of accelerated construction over conventional construction method by an in-depth evaluation of cost, as well as other non-technical factors, despite the higher initial construction costs. This process includes a sensitivity analysis to determine the effect of a change in each factor considered on the identified alternatives.

The Oregon Department of Transportation, together with Departments of Transportation from seven other states and the Federal Highway Administration, developed a decision-making tool based on AHP (Saeedi et al. 2013). The tool is used to determine the most effective bridge construction method from a set of different alternatives for a specific project in the early planning stage where the alternatives include both the accelerated and conventional bridge construction methods. This tool facilitates decision making for any type of bridge project, and it helps to justify the use of a specific bridge construction alternative for a specific project. The tool consists of two level of the hierarchy. The first level of hierarchy consists of direct and indirect costs, schedule constraints, site constraints, and customer service. Each of these criteria is further divided into sub-criteria on the second level of the hierarchy. Further, a detailed definition list of all the criteria and sub-criteria is given based on the experience of the research participants for a proper understanding of the decision hierarchy and consistency of the pairwise comparisons. The pairwise comparison values are entered in a matrix form, and each element of the matrix is divided by the summation of the column to obtain the normalized value. Then normalized Eigen-values are determined by calculating the average of each row, which represents the priorities of each sub-criterion. The pairwise comparison is carried out within the criteria, sub-criteria, and bridge construction alternatives separately to determine the level of preference. Finally, for each alternative, the criteria priorities and alternative weights are then synthesized to obtain a dimensionless value called utility level. The alternative with the highest utility level is selected as the best alternative. The tool is tested and validated using a few case studies, and at the same time, AHP decision-making software is developed.

From the literature review, it has been found that the Federal Highway Administration, together with various State Department of Transportations, has developed high-level decision-making tools to determine the effectiveness of ABC over traditional bridge construction for replacement or rehabilitation of bridges.

### 2.2 PRIMARY FACTORS INFLUENCING THE DECISION ON ABC

The Federal Highway Administration (FHWA 2005) has identified the reduced traffic impact as the major reason for using ABC. Besides this, the issue of site constructability, long detour routes, the requirement of costly temporary structures, bridge location, and limited construction period are other reasons for selecting ABC over conventional bridge construction (Culmo 2011). Hyzak et al. (2013) in their case study indicated the high volume of heavy truck traffic and lack of alternate routes for heavy trucks as major factors for considering the accelerated construction of Gageby Creek Bridge in rural Texas. The authors demonstrated the use of analytical hierarchy decision-making software developed by FHWA to select between the two alternatives, ABC and conventional construction for that project, even though the software was not used during the actual decision. Among the five high-level criteria considered in AHP decision-making process (direct and indirect costs, schedule constraints, site constraints and customer service), customer service received the highest weight, which forms the basis for selecting ABC for that project. Further, the additional cost for the road users resulting from the traffic delay due to bridge construction has been identified as the major cause for the transportation authorities to implement ABC (Khaleghi et al. 2012).

The schedule benefit by using precast concrete elements, weather constraints, and environmental sensitivity are other factors leading to the selection of ABC for the bridge replacement project (Banks et al. 2015). For the emergency
replacement of a collapsed bridge, spanning over the Skagit River in Washington, the Washington Department of Transportation implemented a design-build method for rapid construction using the precast prestressed concrete girder system to minimize traffic disruption (Vanek et al. 2015). Similarly, the Wisconsin Department of Transportation opted for a prefabricated bridge system, which significantly decreased on-site construction duration, for rehabilitation of bridge projects to minimize the traffic flow disruption (Carter et al. 2007). It helped to improve the work zone safety and minimized the environmental impact as well. The Massachusetts Department of Transportation used precast concrete segments to accelerate the rehabilitation of the historic Anderson Memorial Bridge to minimize the traffic disruption, which also helped to replicate the architectural details of the historic bridge in the new one and fulfilled the requirement of passage of boaters under the bridge during construction (Harrington et al. 2014). The Utah Department of Transportation implemented ABC under Highways for LIFE (HfL) program of FHWA in the 4500 South Bridge over I-215 in Salt Lake City and identified three major benefits of ABC technique: minimization of traffic disruption, improved safety for the workers and the road users, and improved quality (Ardani et al. 2010). Further, the economic analysis displayed about 36% cost savings while considering the road users cost saving even though the actual construction cost with ABC exceeded that of the conventional bridge construction method. The Department of Rural Roads in Thailand implemented ABC techniques using a precast segmental approach to achieve higher quality bridges after difficulty in maintaining necessary quality of the on-site concrete casting work in remote rural areas, resulting in poor performance of bridges ( Bamrungwong et al. 2010). The New Jersey Department of Transportation implemented ABC with precast elements for replacing the Route 70 Bridge over the Manasquan River to meet the requirement of coordination between the highway and waterway navigation, and environmental in-water work restrictions (Yermack 2007).

From the above case studies, the primary factors influencing the decision-making of ABC are summarized below:

- Traffic impact
- Site Constructability
- No/long detour routes
- Bridge location
- Limited construction period
- Heavy traffic
- Road users additional cost due to traffic delay
- Weather constraints
- Environmental constraints
- Environmental impact
- Emergency replacement
- Work zone safety
- Historic bridges, replicate the architectural details
- Waterway navigation
- Quality requirements

3 RESEARCH OBJECTIVE

The primary objective of this research is to develop a business case process for ABC considering various drivers as identified from the literature review. The business case process will assist the decision makers in easily deciding on implementation of ABC early in the initial decision-making phase.

4 RESEARCH METHODOLOGY

The first step in this research is to define the research objective. Then an extensive literature review is conducted to determine the gap in research in the field of developing a business case process for ABC. A set of main factors responsible for considering ABC techniques for construction of new bridges or replacement of old bridges are also listed from the literature review of various case studies. Then, a business case process flowchart is drafted based on the literature review and the factors responsible for considering ABC. Further, to learn about the major factor in deciding to undertake ABC for a specific project, industry experts who have been involved in the planning and execution of ABC projects are given a survey questionnaire related to the implementation of ABC, using an online research survey software called Qualtrics. The survey questionnaires are prepared based on the literature review. The interviews are conducted with the industry experts who have been involved in the ABC projects. Finally, based on the survey responses, the proposed process flowchart is finalized that would be very useful for the decision makers to decide whether to select ABC or not for their specific bridge project.
5 RESULTS

For the purpose of determining the prime factor for selecting ABC over conventional bridge construction method, a survey questionnaire based on the literature review is sent to representatives of five state DOT engineers, five contractors, and two fabricators of precast concrete units. However, responses to the survey are received from only one representative from a state DOT (owner) and one representative from a contractor. Both respondents indicated minimizing impact to traffic/traveling public as the main basis for the execution of ABC. This causes a reduction in the construction schedule. Further, both respondents responded no cost savings on the ABC projects based on their experience.

Finally, based on the literature review and survey questionnaire responses, this study proposed a business case process flowchart for the implementation of ABC as shown in Figure 1. The flowchart is intended to produce a decision about the use of ABC for a particular bridge project in the initial decision-making phase. Based on survey responses that the selection of ABC relies on whether there is a reduction in construction schedule or not and there is no cost savings on the ABC projects, schedule saving has been considered as a prime factor for implementing ABC. However, the cost factor has not been considered at any level on the business case process flowchart for selecting ABC over the conventional method.

![Business Case Process Flowchart for Accelerated Bridge Construction](image)

Figure 1. Business Case Process Flowchart for Accelerated Bridge Construction

The first step in this business case process flowchart for accelerated bridge construction is the site reconnaissance by all the concerned stakeholders to gather actual information related to the bridge and the project area. In this step, the information such as the extent of usage of the bridge (traffic volume and type of traffic), location factors (like environmental impact and environmental constraints, weather constraints, waterway navigation, conservation area, etc.), economic impact (community benefit, impact to local economy), availability of detour routes and length of the
detour routes, etc. are gathered for the specific bridge project. Based on these bridge-site specific and surrounding area information, it is determined if ABC is justified for the specific project under consideration or not.

If the selection of ABC is justified based on the information collected, then the next step is to determine the type of ABC to be implemented for the project. It can include either using the off-site prefabricated bridge elements or sliding the completed bridge from off-alignment beside/nearby the bridge location to the final position. Otherwise, the conventional bridge construction option is selected. Once the ABC technique is determined, then the work breakdown structure for the bridge construction process is finalized, and at the same time, the construction work sequencing is also decided. Then the next step is to compare the schedule between ABC and conventional construction method. If the comparison shows an improvement in schedule due to the implementation of ABC, it is decided to select ABC for the project. Otherwise, the conventional method of construction is adopted in the project.

6 CONCLUSION AND RECOMMENDATION

The primary purpose of this research is to develop a business case process for implementation of ABC on any bridge rehabilitation or reconstruction project. Thus, this study proposed a business case process flowchart for accelerated construction of bridges. The proposed flowchart considers schedule savings as a major basis for selecting ABC over conventional bridge construction. The flowchart will be beneficial to all the concerned transportation authorities to select between accelerated and conventional methods of bridge construction for any bridge construction or replacement/rehabilitation project early in the decision-making process. It will help in achieving an optimum benefit from the implementation of ABC.

The proposed flowchart is based on the survey questionnaire responses from two industry experts only, and it considers schedule savings as the prime factor for decision-making. Thus, an extensive survey with more industry experts including representatives from the owner, contractor, and fabricators in association with the transportation agencies like TRB, AASHTO, FHWA, and state DOTs is recommended to develop an improved version of the business case process flowchart. Also, there is a need of further research on developing a business case process tool for implementation of ABC similar to the one developed by the Construction Industry Institute for industrial sector modularization. The business case process tool will be helpful to determine percent modularization of the bridge along with the value of savings as well in comparison to the conventional bridge construction.

REFERENCES


# IRF GLOBAL R2T Conference
November 7-9, 2018 – Las Vegas, NV USA

## PAPER TITLE
Seven California ABC (Accelerated Bridge Construction) Techniques Using Equipment to Move Bridge Superstructures

## TRACK

## AUTHOR (Capitalize Family Name)

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## KEYWORDS:
Accelerated, Bridge, Construction, SPMT, superstructures

## ABSTRACT:

Seven California ABC (Accelerated Bridge Construction) techniques using equipment to move bridge superstructures or falsework components case histories are analyzed. The Maritime Off-Ramp Curve Orthotropic Bridge was installed as 13 sections or as 13 moves via twin truck towed trailers. The YBI (Yerba Buena Island) football field sized post-tension box girder concrete superstructure was installed via skidding jacks. The YBID (Yerba Buena Island Detour) was created via a complex skidding jacks’ operation by removing the existing double decker 66.142-meter Warren truss to falsework for future demolition. Next the replacement truss was skidded to create a revised alignment. The existing steel plate girder Jerrold Avenue Railroad Bridge was removed and a replacement superstructure was installed via SPMT. Railroad steel truss spans were installed via at Highgrove Avenue I-215 via SPMT. The Pfeiffer Canyon superstructure was launched via rollers. The Gerald Desmond Bridge steel falsework was moved via SPMT.
Seven California ABC (Accelerated Bridge Construction) Techniques Using Equipment to Move Bridge Superstructures

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1 INTRODUCTION

This paper will discuss seven case histories which occurred from 1992 to 2018. California has about 12,050 bridges owned and managed by Caltrans (California Department of Transportation). About another 12,030 bridges are owned by counties, cities, the Federal Government and others in California. Belgian, Dutch and German organizations’ patented equipment has been used to move a variety superstructure types in California. There are about 650,000 bridges in the USA. Today there is at least one North American organization with similar equipment. Three European organization’s equipment is slowly being utilized in the USA to move more bridge superstructures via land as an ABC (Accelerated Bridge Construction) process (Mistry & Mangus 2006). The collapse of the FIU (Florida International University) Pedestrian Bridge after moving via SPMT (Self Propelled Modular Transporters) as an ABC choice or process has many questioning this technique as an effective solution. Most of the critics are not in the construction industry. These case histories are presented in chronological order, since lessons are learned from previous projects.

2 CASE HISTORY -1: MARITIME OFF-RAMP OAKLAND, CALIFORNIA

The first curved orthotropic bridge in North America is the Maritime Off-Ramp (Caltrans Bridge No 33-0623s) (Mangus 2003). It was part of the I-880 Cypress Viaduct Replacement Contract “E”. It is the last exit ramp on I-80 prior to crossing the SFOBB (San Francisco Oakland Bay Bridge). It was designed as a curved orthotropic box as it is “horseshoe” shaped to minimize the grade for trucks on a restricted peninsula. Tudor Engineers was the lead designer and engineer of record (Mangus 2014). Caltrans performed peer review for Caltrans (Roberts & Mangus 2004). The bid documents allowed the successful contractor to launch this bridge or erect in large sections this complex bridge. The low bidder chose to erect in large sections.

![Bridge section](image1)

Figure 1. Towing the orthotropic box girder via the “Gizmo” (courtesy & Copyright Caltrans)

Expertise was limited to a few fabricators who could create the welded steel box curved girder with complex geometry. This two-lane bridge was fabricated in 13 sections by Universal Steel in Oregon for the general contractor joint venture Kiewit + Marmolejo. These section lengths varied from 40.234 to 66.750 meters long x 17.374 meters wide [due to curve] and weight from 235 to 416 metric tons in weight. Four sections at a time were...
transported via ocean going barge from the Columbia River in Oregon to the Port of Oakland near the bridge site. Shaughnessy & Company was retained to be the heavy haul manager subcontractor. The decision was to use a “Gizmo” comprised of two clusters of Scheuerle hydraulic trailers paired with steel spreader beams rather than cranes to erect the sections at night (Figure 1). Scheuerle’s patent was issued on 3-18-1985 (Prechtel 1985). The hydraulic trailer patent is the basis for the SPMT (Self Propelled Modular Transporter). The German company’s products web site is [https://www.scheuerle.com/products/modular-vehicles-for-road-transportation.html](https://www.scheuerle.com/products/modular-vehicles-for-road-transportation.html) Four trailers were attached side by side to form a cluster about 7.010 meters wide. Each trailer has six axles with four tires per axle. Thus, each cluster had [4 trailers, 6 axles, 2 pairs, two wheels =] 96 wheels per cluster. A total of 192 wheels where used to move each box girder. The hydraulic wheels ensure that uneven ground surfaces and pot holes can be traversed without shifting the load. Scheuerle hydraulic trailers were used to move the sections from the barge to the staging area near the bridge. The sections were set on blocking to obtain an approximate super elevation. A pair of vertical jacks facilitated the precise fit-up to install about 2,000 bolts at each girder splice. Figure 2 shows a photograph of the installation of the second section with the “Gizmo” as nicknamed by the contractor (McFall & Murphy 1997), (Unknown 1997).

![Figure 2. Front view of towing the orthotropic box girder with the “Gizmo” (courtesy & Copyright Caltrans)](image)

3 CASE HISTORY - 2: YBI VIADUCT

The Yerba Buena Island (YBI) Viaduct [Caltrans Bridge No. 34-0006] carries I-80 traffic across YBI Island and links the East Spans of the SFOBB (San Francisco Oakland Bay Bridge) with the YBI Tunnel. Building a new CIP/PS Box Girder with transverse girders and large edge beams spanning column supports was selected over retrofitting the existing 1930’s era bridge. Caltrans Engineers decided and required in the contract bid documents that a roll-in or slide-in process was required by the low bidder (Kaderabek et al. 2015). The more seismically robust superstructure would be built to the south on columns and falsework that would be left behind. The project replaced the 1930’s era structure by demolishing an existing approach viaduct the size of a football field. This took place on the 3-day Labor Day weekday in 2007. The SFOBB has an ADT of 250,000 vehicles. Caltrans bridge design engineers based in Sacramento were responsible for delivering the structure plans, specifications and estimate for the YBI Viaduct Roll-In Replacement (YBI Viaduct) (Chung et al. 2008). The bid documents only allowed a 72-hour closure, with financial incentives for quicker completion and financial penalties for exceeding the time limit.

![Figure 3. Mammoet Skid System equipment used to slide-in the YBI Viaduct (courtesy & Copyright Mammoet)](image)
The low bid general contractor was CC Myers with Mammoet selected as the heavy lift subcontractor. Mammoet or [Mammoth in Dutch] is a specialty contractor that moves heavy objects that are part of the world’s infrastructure. Less than 5% of their business is moving bridges or bridge components. Mammoet utilizes several types of moving systems. Mammoet’s MSG Skid System is a proven system comprised of an integrated one horizontal jack plus one vertical jack system that is used to move via skidding on steel tracks with cogs as shown in Figure 3 & 4. Mammoet has moved various types of heavy objects using the MSG Skid System. For this move they used sixteen 600 metric ton lifting jacks (Unknown 2007). Thus, there were eight skid shoe rails evenly spaced along the 106.07-meter-long viaduct which weighed 6500 metric tons. Thus, each jack lifted 406 metric tons. The bridge superstructure was first built on 5.029-meter-high temporary columns. Mammoet used manufactured cast steel pipe shims, plus cradles between the bottom of the concrete viaduct and the top of the steel jack. The jacks have a limited vertical movement of 600-mm, so most of the support height was accomplished via shims stabilized with spreader beams plus diagonal bracing. It took Mammoet about 2.5 hours to skid the new viaduct northward into position as shown in Figure 5. Traffic was opened eleven hours earlier than planned (Mangus et al. 2017).

Figure 4. Mammoet Skid System equipment moved the YBI Viaduct (courtesy & Copyright Mammoet)

Figure 5. Mammoet Spreader bar connects Skid System for YBI Viaduct move (Copyright Mammoet)

4 SUMMARY OF THE DOCUMENTARY DISCUSSING THE ABC TECHNIQUES UTILIZED AT YBI

A documentary was created by www.PECG.org that represents the interests of Caltrans Engineers. The goal of this documentary was to clearly explain to the average taxpayer the decisions use to select this successful ABC technique. Related issues are also discussed in this film which has been shown on various PBS (Public Broadcast System) Television stations. The documentary entitled “A Span in Time” was directed by a very experienced director (Brown 2007). The 3-D animation was created by Rick Pepper (Figure 6). The director had cartoon animation by Charlie Canfield to gain the interest of non-engineers. The documentary is told from the integrated combined perspectives of the contractor, dedicated Caltrans engineers and designers, as well as reporters who covered the story. The contractor’s interviews are featured throughout the 28-minute-long video. This film also does a very good job of relaying the sincerity and dedication of Caltrans employees at work in a high stress situation. If the contractor failed then the
most important bridge in California would be shut down. The Caltrans Director, the Toll Bridge Construction Manager, and the Caltrans bridge designers explain many key issues. The Caltrans Director and public relations staff members also explain various methods utilized for this public outreach effort alerting taxpayers, that the SFOBB would be closed for 72 hours for construction. Residents in the area were contacted along with the news media, stakeholders and a visiting out of state college football team plus their out of state traveling fans. This film shows Mammoet performing the skidding or roll-in of the replacement span. The California Highway Patrol managed the command center to shut down traffic prior to the start of the project.

Figure 6. Erection sequence from ABC Documentary (Copyright PECG)

The Caltrans State Bridge Engineer describes the issues involved with this ABC application in further detail. The Documentary’s Director used skillful blending of comedy with the tension of getting the job done while under the media’s microscope. The documentary “A Span in Time” won an Emmy for Outstanding Graphics and Animation in 2009. The technical details are instructive in this documentary plus there is a time lapse sequence of the slide in process. This documentary can be shown in any college course to clearly demonstrate why Accelerated Bridge Construction is utilized. This documentary makes this a captivating experience that illustrates how all involved must work together to create a very successful Accelerated Bridge Construction project. Movie clips may be viewed for free at http://www.aspaninftime.com/ and the DVD can also be purchased. The movie is much more entertaining and instructive than raw footage with narration by the video photographer uploaded onto YouTube or other web sites. Quite a few YouTube videos that are posted on line are just set to music, and there is no narration or verbal explanation of the technology utilized or related issues.

5 CASE HISTORY 3 - YBI DETOUR

The SFOBB YBI Detour aka South-South Detour (SSD) aka East Tie-In structure consisted of a 0.518 km long temporary detour from the Yerba Buena Tunnel to the existing east span of the SFOBB. The SSD was a temporary double-deck detour structure that allowed traffic to cross the existing SFOBB while completing the permanent tie-in structure of the new East Span at Yerba Buena Island as shown in Figure 7. The detour structure was built south of the original bridge and the replacement or new bridge is on the north side. The temporary bypass structure including the East-Tie-In was originally awarded as a “Design-Build” contract to optimize the cost. Caltrans realized in 2007 that the East-Tie-In portion, due to its complex nature and seismic concerns, was best designed in-house. The project was separated from the rest of the design-build viaduct and handled as a capital project. To erect the structure during the 2009 Labor Day Weekend, finalizing of the design often overlapped the fabrication, the bridge designer, prime contractor, erectors, fabricators, and SMR (Structural Materials Representative). The RIRO (roll-in/roll-out) or lateral slide activities were not flexible. Thus parallel design-fabrication activities were required. There was unsurpassed coordination with all parties including designer, detailers, fabricators, erectors, inspectors and other construction support teams. The East Tie-In structure consisted of a new 80-meter single span double deck truss and a skid bent
system. The 2009 Labor Day weekend RIRO process, transferring the double decker 5 lanes EB (lower deck)+ 5 lanes WB (upper deck)of SFOBB traffic to the SSD structure was complex. It involved the removal of the existing double decker steel truss span and installation of the new steel Warren truss steel span on top of a very large steel falsework system comprised of twin skid beams supported by several skid bents plus the four skid towers. AMC Consulting Engineers, Inc. was responsible for the dynamic, static pushover, and construction sequence analyses of all impacted existing and new structural systems. Extensive use of BrIM models was utilized for the design and construction of the East Tie-In structure of the SSD of the SFOBB (Abbas et al. 2014). This structure presented a unique challenge in terms of bridge engineering. It required the replacement of the 91.44-meter-long existing double-deck bridge truss with a similar new truss in a 72-hour window through a RIRO process. It took place about 45-meters above the ground. The YBI detour was the separation of a 91.44-meter-long, 2903-metric ton truss section and the subsequent moving northward of this section of this bridge span. Afterwards, a new 82.296-meter long, 3266 metric ton bridge section was pushed laterally via the Mammoet Skid System into the resulting gap to complete the detour section as shown in Figure 9.

This work was performed on Labor Day weekend in 2009. Mammoet’s MSG skid system had been a proven solution to Caltrans on the adjacent YBI Detour in 2007 (Unknown 2007) (Chung et al. 2008). For this move Mammoet used sixteen Skid Shoes. Each Skid Shoe had a 600 metric ton vertical lifting jack (Figure 4). Each truss chord gusset plate had a pair of skid shoes (east and west sides of gusset plates). Timelines for the project required an extremely challenging schedule for design, detailing, fabrication, and erection of nearly 4536 metric tons of structural steel. The detour replacement truss weighted 2902 metric tons and 1,633 metric tons of steel were used for the skid beams, skid or falsework towers plus bents. Caltrans and the design joint venture of T.Y. Lin International + Moffatt & Nichol Engineers decided to develop fully detailed BrIM (Bridge Information Models) of the East Tie-In structures during the design phase. The primary objective was to use the BrIMs for a virtual simulation of the construction procedure and to evaluate the sequence for the RIRO to identify and eliminate all possible steel fit-up, geometrical conflicts, and space requirement issues. This successfully eliminated any surprises and subsequent schedule impacts during the actual execution process.

The secondary objective was to use the BrIM models of the new structures to generate shop drawings to facilitate the fabrication and erection of structural steel. This significantly accelerated overall construction schedules. BrIM models for the East Tie-In structures, including the new skid bents, skid beams, support towers, and the new roll-in truss were developed in AutoCAD and Bentley ProSteel 3D. Autodesk Navisworks Review & Simulate software was employed for model viewing and simulation. The ProSteel 3D BrIM models were used to generate structural steel shop drawings for fabrication (Abbas et al. 2014). Detailed BrIM models also were developed for the existing truss spans, including the roll-out truss and the adjacent existing and new structures as well as the viaduct, truss, and cantilever structures of the existing east spans. The structures were modeled as solid 3D elements with complete as-built detail right down to connection details in critical areas. The jacking systems used to move the existing and new spans also were modeled in complete detail to ensure proper fit-up of all components. BrIM models were generated to simulate the critical phases of the construction process step-by-step, as a “playbook” through the 72-hour RIRO window. This simulation provided designers thorough studies of critical details of construction procedures in 3D and allowed the design team to refine and fine tune the structural systems and details. This eliminated potential construction issues that could have resulted in schedule delays. Use of fully detailed BrIMs for the East Tie-In was a first for Caltrans plus the joint venture design team of T.Y. Lin International + Moffatt & Nichol Engineers. Thus the shop drawings were successfully generated.
from the BrIM models, which significantly accelerated, plus accurately facilitated the fabrication and erection schedule. The pre-construction virtual simulation of the RIRO resulted in minimization and elimination of many issues that could have potentially caused schedule delays.

Figure 8. YBI Detour Mammoet SMPT positioning Falsework or Skid-Tower (courtesy & copyright Caltrans)

TMF (Thompson Metal Fab) was the fabricator for the skid beam and skid bents. TMF subcontracted some steel fabrication to Jesse Engineering (State of California 2011). Skid bents (pre-fabricated steel columns) were bolted together and braced with steel box beams. On top of those bents, cross beams were placed on center. Even though this was a temporary support system, it carried significant loads and was extremely critical to the entire RIRO operation as shown in Figure 8 and 9. Thus, the RIRO used by Mammoet were fabricated with the same standards as permanent work. It complied with AWS D1.5 (American Welding Society D1.5 Bridge Welding Code) and was inspected as fracture critical components (Wehba & Mehta 2014) (State of California 2010). The skid bents and towers columns were the American Petroleum Institute’s Specification for Line Pipe (API 5L) material X52, that met the ASTM 709 Grade 50 standards. Caltrans brought in Stinger Bridge & Iron in April 2008, to fabricate the new detour double decker steel truss replacement span in Arizona. It spans 82.296 meters between supports. A small temporary dock was built to unload the steel components for site erection on Yerba Buena Island. The detour truss was then successfully shipped from Arizona and erected on top of skid bents. In spring of 2009, Syrinx Industrial Electronics (Netherlands) was contracted by Mammoet. Syrinx specializes in industrial weighing technology and interfaces.

Figure 9. Photo – YBI Detour Mammoet has skid existing truss to the left (courtesy &copyright Caltrans)

This company’s staff was present on-site during the entire project and constructed the measurement system for Mammoet. Syrinx Industrial Electronics used Leutze (Netherlands) ODSL laser sensors to measure distances while moving the five-lane, double-deck toll bridge. Mammoet used 68 sections of pre-engineered or manufactured skid shoe tracks. These prefabricated skid shoe tracks can be field spliced and can be reused (Figure 4). A pair of parallel skid shoe tracks were mounted on both the east and west steel skid beams or falsework. The fabricated weight of the skid beams, skid bents, and skid towers was about 500 metric tons. Alta Vista performed the fabrication inspection of these falsework structures. Mammoet used SPMTs manufactured by Scheuerle to relocate the skid towers after they were assembled on YBI (see figure 8). It took Mammoet 2 hours to lift the existing 82.296-meter long double deck warren truss which weighed 2966 metric tons. Thus, each jack lifted 371 metric tons. The truss was moved about 48.768 meters to the north. Next the newly Arizona fabricated truss was moved in less in a day to its detour
position (3168 metric tons). Thus, each jack lifted about 396 metric tons. Syrinx developed a central monitoring system for this project displayed on a large, 107-cm screen as the two bridge spans were moved. A total of 31 Leutze laser distance sensors and 16 Leutze pressure sensors were used to guide the sliding of both trusses. A robust monitoring design and the display on the ODSL 30, allowed measurement values to be read on the spot. Thus it was not necessary to constantly run back to the central laptop to make adjustments during sliding. This saved considerable time. The Leutze electronic sensors were affixed to mounting plates that were easily applied and positioned through the use of strong magnets to the trusses being moved. Measurement of both small and large distances while moving the truss span during the RIRO, each truss span was pushed upward at four points approximately and then moved laterally by means of Mammoet hydraulic skid shoe system with a horizontal movement or stroke of 1400-mm per Figure 4. Then the hydraulic jack is repositioned between the next set of cogs. The height and position of the various hydraulic skid shoes were converted to 4 to 20 mA measurement signals with the aid of the Leutze electronic ODSL 96B laser sensors. The total horizontal movement of each truss was approximately 30-m and was measured with Leutze ODSL 30 laser distance sensors. Syrinx Industrial Electronics also developed eight measurement boxes to which all laser distance sensors and pressure sensors could be connected. Together, all of the measurement boxes formed the complete measurement network that was connected to a single laptop. This laptop was then connected to the 107-mm monitor, on which the processes could be visualized as it happened. This lateral slide project received an AISC-NSBA Award (National Steel Bridge Alliance 2012) (Copelan et al. 2011).

6 CASE HISTORY 4 – RAILROAD PLATE GIRDER BRIDGE AT JERROLD AVE

The original Jerrold Avenue Railroad Bridge built in 1907 by Southern Pacific Railroad was deteriorating and no longer met modern seismic safety requirements. Caltrain contracted HNTB to assist with replacement of this railroad bridge. HNTB designed a replacement bridge that was installed in a single weekend using SPMTs. A method Caltrain had never used before to address Caltrain’s critical need to minimize disruption to their commuter traffic. Maintaining regular train service was essential to Caltrain, who transports 35,000 average weekday commuter passengers into San Francisco along the San Francisco Peninsula. HNTB’s replacement bridge design eliminated the skewed intermediate supports of the original three-span steel plate girder bridge and widened Jerrold Avenue below by using a single, steel span.

![Figure 10. HNTB Concept for Caltrain Jerrold Avenue Railroad Bridge (courtesy & copyright HNTB)](image)

The final design of the replacement bridge is a steel through-girder structure. Twin plate girders each 41.128-m long with 3.962-m deep webs plus 7.62-mm-thick flanges. Four new 2.800-m diameter CIDH (Cast-In-Drilled-Hole) piles were installed by Disney Construction without stopping any train. So one CIDH was placed in each corner plus outside of the existing skewed bridge (Figure 10). Two reinforced concrete bent caps were built at each end connecting pairs of CIDHs with straddle bent caps below the existing bridge. Thus the use of CIDH piles connected with reinforced bent caps allowed the contractor to complete the accelerated replacement without interrupting Caltrain’s commuter traffic along this corridor. The new steel replacement bridge was reassembled, after shop fabrication, near the tracks on a vacant property. The SPMT was invented by Scheuerle in Europe in the 1980s and also patented in the USA (Kalkman & Roodenburg 2000).
Starting after midnight on Saturday October 15, 2011, Disney Construction employees set to work ripping out the 1907 rusted steel bents and existing concrete south approach. Next the SPMTs had moved away the existing bridge onto nearby temporary cribbing or falsework to await demolition, plus recycling of the steel. Next Disney construction crews and Mammoet moved the new wider bridge into position using additional sets of SPMTs. Truck cranes lowered flanking precast concrete approach spans to complete the replacement bridge. One of the two main parallel tracks was reopened Saturday afternoon, approximately 12 hours after being closed. The parallel second track reopened Sunday afternoon, allowing Caltrain to resume normal commuter weekday service operations for the Monday October 17, 2011 morning commute. The $7 million Jerrold Avenue Bridge Replacement, which came in more than $1 million below the engineer’s estimate, satisfied Caltrain’s tight budget.
This bridge is part of the $205 million project of adding HOV (High-Occupancy Vehicle) carpool lanes north of Orange Show Road in San Bernardino and south of the 60/91/215 interchange in Riverside in Southern California. So 4 traffic lanes in each direction were widened to six lanes in each direction. The existing BNSF (Burlington Northern Santa Fe Railroad) Railroad Bridges (Caltrans Bridge No. 54-1305) cross over I-215, from the City of Grand Terrace to the City of Colton. Caltrans partnered with Ames Construction Inc., Stinger Bridge & Iron, San Bernardino Associated Governments (SANBAG), Riverside County Transportation Commission (RCTC), The Federal Highway Administration (FHWA), BNSF, and the California Highway Patrol to facilitate utilizing the best ABC solution. The existing railroad structure was a four-span steel through plate girder bridge carrying two railroad tracks. Thus there was an east bound and west bound track for BNSF train traffic. This limited the I-215 freeway width of twelve lanes under the railroad bridge. To accommodate the additional HOV lanes adjacent to the center line median, the bridge piers would have to be relocated between traffic lanes. Thus a safety hazard would be created by dividing parallel traffic lanes. Additionally, the existing column Bent 3 was inadequate for the increased design loads associated with longer spans. Thus, it was determined that the most practicable solution was to completely replace the existing railroad bridge with new longer spans. Thus Highgrove Underpass, owned by BNSF, was replaced with longer spans plus fewer intermediate support columns or piers.

A two-span steel truss railroad bridge was selected to eliminate the columns which divided the parallel six lanes as 2 HOV + 4 regular lanes as they passed under the existing plate girder railroad bridge. First, a detailed bridge type selection was carried out, and twin two-span steel truss bridge with span length 60.92-m (total 121.92-m) and 20-degree skew was selected from three other logical bridge alternatives.

Railroad bridges are functionally and behaviorally different from highway bridges (Wu et al. 2013). Historically, railroad bridges have performed well in seismic events with little or no damage. AREMA provides seismic design guidelines for railroad bridges. The replacement truss, designed by Caltrans engineers was fabricated from weathering steel, which will require no painting. Painted bridges require future painting, so BNSF switched to weathering steel for the replacement bridge. The existing railroad bridge was painted a silver color. The general contractor Ames Construction with subcontractor Stinger Bridge & Iron proposed a Value Engineering solution to move the permanent trusses twice in a “musical chairs” sequence. BNSF had been thinking in selecting traditional shoofly bridges, which are built, used for a few years and then demolished. The ABC solution relocating the same truss spans twice via SPMTs was approved by the stakeholders. First, the four of the six truss spans would be installed in the shoofly west bound and shoofly east bound track alignments using SPMTs. Afterwards, these same four truss spans would be relocated to their final east and west track alignment. This final move occurred after the 1950’s era plate Girder Bridge was removed and new concrete piers and piers were constructed (see Figure 13)
This eliminated building an extra bridge that would be demolished after less than three years of usage. Stinger Bridge and Iron shop fabricated the trusses in Arizona. The truss pieces were shipped to California. Ames Construction assembled the 499 metric tons trusses about 1.2-km away at the “Bridge Farm” site (Chung & Wu 2016). The steel trusses were connected together with 98,000 bolts. This limited the transport distance via SPMTs owned by www.Sarens.com California to their installed locations. This Belgium Company’s USA office is based in Los Angeles. Sarens gained very positive California publicity by moving the Endeavor Space Shuttle via SPMTs to its California museum location. These trusses were then lifted and checked for logistic capabilities. Finally, the four 60.96-m long, 9.144-m high and 7.01-m wide steel trusses were transported into place using SPMT (shown in Figure 14). Sarens used equipment manufactured by Scheuerle and paints them blue. Mammoet uses the color red for its equipment. Sarens has blue painted steel cradles and shims to fill in the gap between the top of the SPMTs and the bottom of the steel truss bridges. The replacement had to be done in a 20-hour window because it required shutting down one track at a time to keep rail traffic moving. Sarens transported four steel railroad bridge trusses using SPMTs at a Saturday midnight moves via I-215 into place with minimal traffic impact to I-215 motorists. Sarens used it equipment on other client’s projects between truss moves. The last spans were set in September 2014 (shown in Figures 15 & 16). The complex series of “musical chair” moves is described in much greater detail in the Caltrans DES Education Committee presentation (Chung & Wu 2016).

8 CASE HISTORY 6 – PFEIFFER CANYON STEEL BRIDGE LAUNCHING

Pfeiffer Canyon Bridge (Caltrans Bridge No. 44-0060) was completed in 1968 and was a three-span continuous concrete curved box girder bridge with two piers in the canyon. It has a total length of about 95 meters. It was located near Big Sur of Monterey County on Highway 1. After heavy rains on February 11, 2017, an individual walking under the bridge noticed cracks in one of the columns and saw the ground under the concrete bridge washing out from underneath the footings. Caltrans was notified and restricted its usage. The concrete bridge remained open to pedestrians for a period and Caltrans allowed one vehicle at a time over the bridge during two periods on Feb. 15. During the week of April 11, 2017 Caltrans retained a demolition contractor to demolish the concrete bridge. They used a wrecking ball and other equipment to strike precise points on the bridge to break the concrete bridge apart into smaller pieces. A rapid emergency design of a single span bridge utilizing three parallel steel plate girders was based on the successful Whites Hill Sidehill Viaduct (Caltrans Bridge No. 27C-0157). This similar simple weathering steel span bridge owned by Marin County had successful service since 2003 (13+ years). The simple straight steel span design of about 95-meters eliminated intermediate columns and structural vulnerability from future landslide activity in the canyon floor or valley. By requiring an ABC method, Caltrans had an operational replacement bridge in seven months when a normal delivery project of similar size plus cost would typically take 2 to 3 years.

![Figure 17. Three plate girders were installed via launching across Pfeiffer Canyon (copyright Caltrans)](image)

The new structure (Caltrans Bridge No. 44-098) was fabricated as 1 steel girder segments in Vallejo, CA weighing 56.245 metric tons each that span the rugged, 94.488-foot canyon. The new replacement bridge was designed by Caltrans bridge engineers and detailers in Sacramento. This steel bridge has a straight alignment with 3.657-m wide lanes. The 1.524-m wide outside shoulders make it accessible for all modes of transportation (pedestrians and bicyclists). Caltrans ordered the steel in advance via XKT Engineering fabrication plant on Mare Island in Vallejo, CA. This accelerated the design and used an innovative way to pull the assembled steel girder bridge into place (Wilcox & Smith 2018). The prime contractor for the $24 million Pfeiffer Canyon Bridge emergency replacement project was Golden State Bridge Company of Benicia, CA. The ABC “bolt on launching nose” plus construction processes were designed by a specialty engineering firm subcontractor to Golden State Bridge Company. The Accelerated Bridge...
Construction technique of launching via Hillman Rollers and pulling cables was used to install the plate girders. These three girders were pulled upward via cables from the lower abutment 2 to abutment 1 which is at a slightly higher elevation.

![Image of construction technique](image1)

**Figure 18.** Three plate girders were lowered launching across Pfeiffer Canyon (copyright Caltrans)

The contractor erected a temporary center support pier to facilitate the launching. The center pier was manufactured scaffolding painted red on a concrete footing. After the launching, then the three girders were lowered via hydraulic jacks to their final elevation (Figure 18). Dozens of bridge workers worked long hours, mostly 6-7 days a week from the end of March 2017 to complete the replacement bridge. On October 13, 2017 Caltrans, along with state leaders and local partners celebrated the completion of the State Route 1 Pfeiffer Canyon Bridge project, just eight months after this bridge was damaged by harsh winter storms (Figure 19). Some final work, such as deck formwork removal, was performed after the bridge opened to traffic. The new bridge restores a vital link between Big Sur and the other communities in Monterey.

![Image of public opening ceremony](image2)

**Figure 19.** Public opening ceremony across Pfeiffer Canyon; notice deck falsework remains (copyright Caltrans)

### 9 CASE HISTORY 7 – GERALD DESMOND BRIDGE MSS FALSEWORK

The Gerald Desmond Bridge Replacement project is designed to cross a Port of Long Beach channel between Terminal Island and the remainder of Long Beach with a much greater vertical clearance for PANMAX shipping vessels. It has a cable-stayed main span over the waterway channel and series of 45.72-m long concrete approach spans. The contractor Shimmick Construction purchased two specially engineered and fabricated movable falsework or scaffolding system to build the parallel approach bridge spans. The technical name for this equipment is a self-launching MSS (Movable Scaffolding System). The cost was about $20 million total or $10 million for each MSS, which is fabricated in Europe of welded steel. Each assembled MSS or falsework or movable formwork weighs about 1361 metric tons of steel when completed. One MSS was shop painted blue and the second MSS was painted orange. This European developed
technology or MSS has never been used to build a bridge in California, according to the design-build team. The two MSS were shipped from Europe in many pieces and assembled in a vacant lot area near the bridge. This MSS system eliminates the need for workers to build the traditional wood plus steel site constructed falsework. This pre-engineered solution was an ABC solution selected by the contractor. They claim that there are significant cost plus time savings.

Figure 20. MSS Falsework moved via SPMT for Gerald Desmond Bridge (copyright Port of Long Beach)

Next, the MSS was moved to the intimal approach span location via SPMT. Next it was lifted vertically using cables from the previously completed concrete bridge pier. The MSS is anchored to columns supporting the superstructure. The MSS Falsework can be moved about 71.628 m outward from its starting position to allow construction crews to begin the next approach span. Skilled construction employees can access areas below the bridge to use laser surveying equipment to ensure the scaffolding system and segments of the future bridge are accurately aligned. After the concrete for each segment has been placed, workers can lower the steel formwork and begin extending the movable falsework system forward or outward into space so crews can begin the next approach span of the bridge.

Figure 21. Operator Controls SPMT for moving Gerald Desmond Bridge MSS (copyright Port of Long Beach)

The project’s engineering and safety requirements restrict the scaffolding system’s movements to about 600 cm at a time. It takes about one work day for the MSS to move the entire distance needed for workers to separate from one completed approach span to be relocated begin the construction of adjacent or next approach span.

10 CONCLUSIONS

In the USA, the low bidder is normally selected to build the project or bridge via a sealed process. After the end of the bid deadline, these sealed bids are opened and read in public “bid opening” event. The low bidders bid is checked by the owners staff for any errors. A successful bidder is awarded the contract. The owners engineer and or design engineers, such as Caltrans can set parameters requiring an ABC solution. Parameters can eliminate cast-in-place construction and require the moving of a superstructure either by any means or method. The expanding use of ABC techniques will make them more common place. After patents expire then additional manufactures can legally produce SPMT or rollers etc. This results in drop of the sales price of ABC equipment due to competition between
manufacturers. Once equipment is purchased then the owner or contractor will also want to continue to use it, as much as practicable. Currently these ABC techniques described in this paper are very rare utilized in North America. The organization PECG (Professional Engineers in California Government) created three ABC documentaries shown on PBS TV stations, plus professional papers describing successful ABC solution case histories for various technical conferences to inform the other industry professionals (Brown 2006), (Brown 2007), (Brown 2008), (Kaderabek et al. 2015), (Mangus et al. 2017), & (PBS 2008). Every construction method has a risk and reward. Studying failures may only demonstrate that some key issues were not properly considered. The payloads continued to increase and are basically unlimited today. Today, 70% of all transports over 3,000 metric tons and 90% of all transports over 5,000 metric tons are performed on SPMT vehicles manufactured by Scheuerle. Thus, SPMT has maintained a position in the oil and gas industry, shipbuilding, and many other construction sectors. Unit weights of 14,000 metric tons, a world record, have already been transported with SPMT, manufactured by Scheuerle. The primary idea of quickly erecting a structure when the freeway or city street was only partially followed by the FIU (Florida International University) Pedestrian Bridge design build team. This tragic failure is being widely studied in the media, taxpayers and industry professionals. The first of two bridge spans portion was site cast in a vacant lot. Then it was rotated at night 90 degrees via SPMT with local street closure on a Saturday midnight. The design-build team made some error(s) that resulted in this small bridge collapse, killing five uninvolved citizens stopped below this bridge, awaiting a traffic signal red light. These commuting individuals were not properly restricted from driving below this dangerous phase of stressing the concrete truss chord. Many may see that this truss chord post-tensioning jacking could have been performed during another Saturday night closure. The issues and details described in this paper are based on press releases and sales brochures giving a marketing overview of the basic practicable choices that are available. Thus trade secrets and “how to do it” are not really public domain knowledge. Other solutions and sub variations may exist. The bridge owner in the USA must allow one or buildable techniques to be available. Thus the power of the owner is to only allow an ABC solution(s) to be bid by the construction community.

11 ACKNOWLEDGEMENTS

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12 REFERENCES


The response of hybrid piles reinforced with glass fiber polymer material in designing deep foundations

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KEYWORDS:
Pile design, fiber reinforced polymer, hybrid piles, bridge design, composite piles.

ABSTRACT:
Designing hybrid piles can enhance the durability and decrease the estimated cost of building and maintenance of deep foundations structures compared to the traditional methods such as concrete and steel piles. This study deals with analyzing the structural behavior of glass fiber reinforced polymer (GFRP) tubes filled with non-structural material as hybrid piles in designing deep foundation systems. Piles reinforced with GFRP material adds the durability of the foundation structures in harsh marine environment such as coastal areas which in respect reduce the cost of maintenance in the long run. By conducting experimental tests on the full-scale samples, the response of the piles reinforced with GFRP tubes is analyzed. Based on the test’s outcomes, GFRP hybrid piles demonstrate semi-linear response. Structural strength and ductility of the GFRP hybrid piles is more than traditional piles, while the axial capacity of this hybrid pile is comparable to the conventional piles in designing deep foundations.
The response of hybrid piles reinforced with glass fiber polymer material in designing deep foundations.

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1 INTRODUCTION

One of the main concerns of using traditional steel, concrete, or even wood piles in bridge structures is their short-term deterioration and structural failure. In the marine environment, where salt water accelerates corrosion, traditional materials are most prone to corrosion in splash and tidal areas. Some solutions, such as using epoxy as a coating layer or galvanizing the steel member, have been employed over the past few decades (Clarke 1999). However, this had unsatisfying outcomes about increasing the lifespan of steel materials in a corrosive environment (Keesler & Powers 1988). Moreover, in the long run, abundant use of steel and reinforced concrete for piles increases the demand for restorations and strengthening (Hong et al. 2010). The short lifespan of these conventional materials can increase the cost of maintenance significantly. For instance, either replacing or repairing these piling systems costs more than one billion dollars annually in the U.S. (Lampo 1996).

In contrast, Fiber Reinforced Polymer (FRP) is corrosion-resistant when compared to steel and other traditional materials. Furthermore, FRP piles can dissipate and absorb the impact energy of ships and other vessels and serve as mooring points. Based on the highly acceptable performance of FRP materials for applications in marine engineering, the use of FRP sheets in construction gained a reputation as a practical solution against corrosion, and also improved the durability of structural members in such a hazardous environment (Tharmarajah et al. 2010). Therefore, fiberglass piles are an ideal material for marine applications.

Hybrid FRP materials, as composite materials, have more merits for designing piles for structures such as bridges compared to non-hybrid materials. For instance, glass fiber reinforced polymer (GFRP) sheets, as a hybrid reinforcing method, have superior features regarding their strength, tensile and stiffness ratio to weight. GFRP is both nonmagnetic and nonconductive which is an advantage compared to the traditional materials (Panda et al. 2010). Also, the constructability of GFRP extends its usability for structures having complex shapes is another of advantage to be considered. In addition, these hybrid FRP polymers are more resistant to hazardous environments when exposed to saltwater or chemical materials or even during destructive hurricanes. Therefore, in specific environmental conditions, such as coastal areas, replacing conventional pile members with GFRP hybrid piles improves the lifespan of the constructed members against corrosion more considerably.

This research examined the behavior of hybrid fiberglass tubes filled with recycled material and concrete for use in deep foundations (piles). In this experiment, fiberglass tubes were filled with concrete containing recycled materials, such as shredded and used tires. The use of recovered materials results in lowering the construction costs in an environmentally friendly approach. In this way, seemingly worthless and useless materials could be used as part of construction process without sacrificing natural resources or producing hazardous material during construction.

Significant parameters should be regarded when reinforcing structural members, such as piles, by using hybrid FRP layers. Choosing the appropriate hybrid material – for example, GFRP, hybrid carbon and glass reinforced polymer (HCG), or carbon fiber reinforced polymers (CFRP) – is one of these important parameters. As another reinforced hybrid option, using Basalt fiber-reinforced polymer (BFRP) sheets is almost 15% less expensive compared to GFRP materials with the same compressive strength as GFRP (Serbescu et al. 2010).

Moreover, features of the hybrid layer – such as its arrangement, thickness, width, and the number of the designated layers as well as its average concrete compressive strength – affect experimental outcomes. Another parameter that should be taken into consideration is specimens’ shape in that circular or squared specimens may result in different outcomes. Hybrid FRP materials, such as GFRP, are composed of either bonded or embedded fibers in a matrix arrangement. This specific matrix pattern transfers structural loads within mediums and reinforces the fibers
against environmental hazards. With this arrangement, the fibers mainly are designed to sustain structural loads (Vatani Oskouei et al. 2010).

2 METHODOLOGY

This study deals with placing GFRP composite piles, based on the winding method, as precast members in circular directions, with concrete or recycled-based materials as fillers. By applying a specific pattern using this technique, polyester material such as resin could be utilized to create the appropriate fibers.

2.1 STRUCTURAL CRITERIA

A GFRP pile was built that had an outer diameter of approximately 600 mm; the accumulative thickness of the member’s wall was about 7.5 mm. The GFRP composite piles included three inner and outer layers, using a fiber-oriented material that was inclined longitudinally almost 40 degrees from the horizontal axis of the pile. The segment between these two layers had an inclination of approximately 80 degrees from the longitudinal axis in the tube. Using expansive additive materials in the tube enhanced the strength of the existing concrete against shrinkage and helped to maintain the compression strength of the member.

2.2 EXPERIMENTAL TESTS ON GFRP PILES

A full-sized pile sample, using GFRP, was built and used in the laboratory to increase the accuracy of the test results. After determining the specific stiffness of the tested tube member, the sample’s strength was calculated by applying axial-loading and moment-loading scenarios.

The performance of GFRP tubes filled with recycled material was evaluated in tensile and compressive loading scenarios. The structures of three different specimens were analyzed, and the loading and bending ultimate capacities were measured. The loading conditions and the type of tube materials, as well as the span height, are presented in Table 1. GFRP piles' specifications such as outer diameters, Poisson ratio, and other measured data are illustrated in Table 2. The GFRP members were designed to be based on the winding method. Considering the longitudinal tension forces in the axial direction of the tested piles, and based on lamination theory, the failure modes of the GFRP piles were analyzed.

Table 1. Experimental Data and Specifications of Samples

<table>
<thead>
<tr>
<th>Eccentricity (mm)</th>
<th>Length of Span (m)</th>
<th>Axial Loading (KN)</th>
<th>Moment Loading (KN-M)</th>
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<tbody>
<tr>
<td>BE 1-1 Tube 1</td>
<td>∞</td>
<td>4.5</td>
<td>0</td>
</tr>
<tr>
<td>BE-AX 1-1 Tube 1</td>
<td>647</td>
<td>1.6</td>
<td>198</td>
</tr>
<tr>
<td>BE-AX 2-1 Tube 1</td>
<td>322</td>
<td>1.6</td>
<td>326</td>
</tr>
<tr>
<td>BE-AX 3-1 Tube 1</td>
<td>239</td>
<td>1.6</td>
<td>312</td>
</tr>
<tr>
<td>BE-AX 4-1 Tube 1</td>
<td>144</td>
<td>1.6</td>
<td>427</td>
</tr>
<tr>
<td>BE-AX 5-1 Tube 1</td>
<td>54</td>
<td>1.6</td>
<td>344</td>
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<tr>
<td>AX**.1-1 Tube 1</td>
<td>0</td>
<td>1.0</td>
<td>677</td>
</tr>
<tr>
<td>BE 1-2 Tube 2</td>
<td>∞</td>
<td>4.5</td>
<td>0</td>
</tr>
<tr>
<td>BE-AX 1-2 Tube 2</td>
<td>547</td>
<td>1.55</td>
<td>446</td>
</tr>
<tr>
<td>BE-AX 2-2 Tube 2</td>
<td>417</td>
<td>1.55</td>
<td>557</td>
</tr>
<tr>
<td>BE-AX 3-2 Tube 2</td>
<td>310</td>
<td>1.55</td>
<td>612</td>
</tr>
<tr>
<td>BE-AX 4-2 Tube 2</td>
<td>189</td>
<td>1.55</td>
<td>643</td>
</tr>
<tr>
<td>BE-AX 5-2 Tube 2</td>
<td>21</td>
<td>1.55</td>
<td>811</td>
</tr>
<tr>
<td>AX-1-2 Tube 2</td>
<td>0</td>
<td>0.97</td>
<td>901</td>
</tr>
<tr>
<td>BE 1-3 Tube 3</td>
<td>∞</td>
<td>4.5</td>
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</tr>
<tr>
<td>BE-AX 1-3 Tube 3</td>
<td>733</td>
<td>1.75</td>
<td>288</td>
</tr>
<tr>
<td>BE-AX 2-3 Tube 3</td>
<td>522</td>
<td>1.75</td>
<td>401</td>
</tr>
<tr>
<td>BE-AX 3-3 Tube 3</td>
<td>319</td>
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<td>522</td>
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<td>BE-AX 5-3 Tube 3</td>
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<td>848</td>
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<tr>
<td>AX-1-3 Tube 3</td>
<td>0</td>
<td>0.94</td>
<td>1002</td>
</tr>
</tbody>
</table>

*: Bending Loading  **: Axial Loading

For drawing bending forces, four stations were checked; for the beam samples, three similar samples were utilized to evaluate the flexural resistance. The beam results illustrated that the total member responses were linear;
therefore, cracking could be omitted when considering higher amounts of loading strength in both flexural and axial loading conditions.

### Table 2. Tested Specifications for GFRP Piles

<table>
<thead>
<tr>
<th>GFRP Type</th>
<th>T-1</th>
<th>T-2</th>
<th>T-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average (E_{\text{Axial}}) (GPa)</td>
<td>14.3</td>
<td>16.6</td>
<td>19.5</td>
</tr>
<tr>
<td>Average (E_{\text{loop}}) (GPa)</td>
<td>15.2</td>
<td>17.3</td>
<td>21.4</td>
</tr>
<tr>
<td>Outer Thickness (mm)</td>
<td>4.31</td>
<td>4.87</td>
<td>5.24</td>
</tr>
<tr>
<td>Poisson Ratio</td>
<td>0.071</td>
<td>0.085</td>
<td>0.092</td>
</tr>
<tr>
<td>Tensile strength* (MPa)</td>
<td>178</td>
<td>201</td>
<td>263</td>
</tr>
<tr>
<td>Compressive strength** (MPa)</td>
<td>164</td>
<td>187</td>
<td>255</td>
</tr>
<tr>
<td>Diameter (mm)</td>
<td>227</td>
<td>236</td>
<td>241</td>
</tr>
</tbody>
</table>

*,**: Both tensile and compressive stresses were measured in axial directions.

Furthermore, the deflection responses of the test beam in the presence of increasing axial loads were evaluated by three beams, BE1-3, BE1-1, and BE 1-2, based on both laboratory and analytical data. Figure 1 illustrates that the samples’ responses before the failing points were linear. Moreover, in all three beam tests, the ultimate load capacity was significantly higher than the calculated load for cracking the samples. Furthermore, some minor data fluctuations were insignificant due to the sample's calibration and initial measuring conditions (Fam et al. 2002).

![Figure 1. Deflection response of the test beams in the presence of the axial load.](image)

3 DATA ANALYSIS

Based on the resulting laboratory data, various responses of the piles tested were analyzed, and their curvature graph was evaluated against the moment reaction of these composite members. The responses of the tube members after cracks were determined, and the overall slip of recycled materials adjacent to the tubes were measured.

3.1 MATERIAL VERIFICATION

Material verification should be controlled by conducting 'coupon' methods as well as applying the lamination theory to calculate the maximum measured stiffness and strength capabilities of the GFRP members (Fam et al. 2003a). The first step was to sample an almost rectangular-shaped strip layer from the tube, which had dimensions of 25 mm and 600 mm, longitudinally. The actual strength measurement of the tube has an error at the very end of both fixed
locations due to the stress concentrations around those regions (ACI 318 Committee 2002). In the next procedure, the steel tubes were used as a cover around the fiberglass tubes in the presence of resin to fill the empty voids. In this step, the GFRP coupons had a free (unfixed) length of 178 mm. After finishing the preparation stage, GFRP samples at the fixed-end positions were tested for tension loading. Figure 2 displays the results of the accumulated strain and stress.

![Figure 2. Comparison of axial stress-strain based on laboratory data, the lamination theory, and an ideal hypothetical line.](image)

### 3.2 RESPONSE EVALUATION OF GFRP AND PRE-STRESSED PILES

The responses of pile members in loading scenarios were based on their material. As a result, various composite materials considered in this research – including GFRP, pre-stressed, and concrete – had different strengths during the loading experiments. Due to this discrepancy, designers of bridge structures use different design methods to deal with various types of pile members (PCI Committee 1993). Based on the responses, in this study, analysis of the pile members was performed by drawing a comparison between the moment-curvature graphs (Daniel & Ishai 1994).

First, the geometric characteristics of the GFRP filled with recycled materials was analyzed based on the following procedure. A filled GFRP tube was striped longitudinally and sampled. From the stress-strain data gleaned from the experimental test, which was semi-linear, the internal forces acting on the filled materials and the GFRP were evaluated by conducting a numerical analysis and using regression on the resultant laboratory data, as illustrated in Figure 3. Second, to develop the moment-curvature diagram, the calculated internal forces were used to anticipate the internal moments in various locations with regard to the estimated strain data. Three tests were conducted to draw a more precise analogy of the moment responses of the GFRP piles.
Based on Figure 3, as the internal moments approached the failure point, the cracking occurred and, the flexural capacity of the GFRP, as a non-hybrid FRP material, has nonlinear behavior and decreased notably. Before the occurrence of cracking, the stiffness capacity of the GFRP tubes and conventional pile is almost identical. Since GFRP piles were filled with recycled materials, instead of reinforcing, the occurred cracks would not affect the lifespan of the members in the long term.

3.3 FIELD RESULTS ON GFRP PILES

As mentioned before, drawing a more accurate analogy of employing GFRP composite piles can be achieved suitably if the tests were conducted on full-scale models (Fam et al. 2003b), (Pando et al. 2000). The capacities of GFRP hybrid piles were measured in axial loading condition and for determining geotechnical strength. Installed strain gauges and accelerometers measured the member's displacements both longitudinally and laterally. The lateral deflection and displacement of the pile as well as the axial strain and displacement responses of the piles were measured and analyzed. Based on the outcomes, the axial capacity of these composite members was considerably higher than their geotechnical capacity.

4 CONCLUSIONS

The outcomes of this research were based on the specific arrangement of GFRP layers, including their diameter, outer and inner thickness, and the inclined degree from the horizontal axial axis. Altering these specifications as well as changing the material’s properties – such as $E_{\text{Axial}}$, $E_{\text{Hoop}}$, and Poisson ratios – changed the results. Based on results from three beam samples, their deflection responses in axial loading condition, before reaching the failure criteria, is linear. Because the recycled material was used as filler inside the tubes, the effect of cracking was negligible, in the long run, for high values of calculated strength and in the presence of either axial or flexural loadings. Some minor inconsistencies in the outcomes were the result of such parameters as initial measurements and the calibration of samples.

Analyzing the internal forces of the GFRP tube filled with recycled material illustrated that the behavior of the member was semi-linear, which resulted in enhancing the total strength and ductility of the reinforced piles. The lateral responses of both the GFRP piles and pre-stressed piles demonstrated almost analogous results in that the effect of cracking in the presence of considerable axial and flexural loadings was negligible.

Future projects may investigate how to add more precision to methods of developing and installing pile members when using GFRP materials, for example by analyzing more sample data. Furthermore, as an ongoing topic for future research, the effects of applying basalt fiber-reinforced polymer as an alternative for GFRP material could be investigated for more cost-effective pile design and implementation.
REFERENCES


**PAPER TITLE**: Numerical Study of the Impact Response of Ultra High Performance Fiber Reinforced Concrete Beams

**TRACK**: Innovative Materials and Technology

<table>
<thead>
<tr>
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<td>USA</td>
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<td>USA</td>
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**KEYWORDS**: Nonlinear impact analysis, Ultra-High-Performance Concrete (UHPC), Reinforcement ratio, Sensitivity analysis, and Accelerated construction.

**ABSTRACT:**
The material and structural behaviors of Ultra High-Performance Concrete (UHPC) under a high strain rate loading are different than a quasi-static loading condition. Though, UHPC has very high compressive strength compared to conventional concrete, the failure strain of UHPC is not sufficient to withstand large plastic deformations under high strain rate loading such as impact and blast loading. Due to the high cost of large-scale experimental research of structural elements under impact loading, modeling of UHPC bridge deck girders using computer aided program has become a need to broaden and enhance the current knowledge. This study is conducted using the explicit finite element (FE) computer program ABAQUS. The numerical study is verified using the experimental results available under impact loads. Five different types of UHPFRC beams were simulated under impact loading to observe the deflection-time response. The key parameters investigated are the reinforcement ratio ($\rho$), impact load under various drop heights ($h$), and the failure phenomena. It was observed that higher reinforcement ratio showed better deflection recovery under impact. Also, for a specific reinforcement ratio, the maximum deflection increases approximately 15% when drop height decreases. The results also provide recommendations for predicting the location of the local damage in UHPFRC beams under impact loading.
NUMERICAL STUDY ON THE IMPACT RESPONSE OF ULTRA-HIGH-PERFORMANCE FIBER REINFORCED CONCRETE BEAMS

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1 INTRODUCTION

Impact loading is an extremely severe loading condition characterised by a force of great intensity within a very short duration. The behavior of a structural component under impact loading includes two response phases, a local response due to stress wave generated within short duration of impact followed by a free vibration effect due to elastic-plastic deformation over an extended period after impact. The overall response depends predominantly on the loading rate effect and the dynamic behavior of the structural component (Fujikake et al. 2009). So far, significant efforts have been provided in developing innovative ways to increase the long-term structural performance and currently, the use of Ultra High-Performance Concrete (UHPC) has drawn the attention for its superior properties such as early high strength gain, extreme durability, low permeability and long-term stability. The term UHPC refers to a class of advanced cementitious composite materials where innovative technology of cement and concrete industry blends together (Graybeal 2010). Therefore, UHPC is considered as a promising material for innovative structures subjected to severe loading conditions such as impact, shock, and explosive loadings (Fujikake et al. 2006).

Some experimental program has been conducted recently focusing on material characterization (Graybeal 2006a), flexural and shear tests of I-girders (Graybeal 2006b, 2008), prototype pi-girders (Graybeal 2009a), and second-generation pi-girders (Graybeal 2009b). Unfortunately, current understanding of the impact and blast resistances of Ultra High-Performance Fiber Reinforced Concrete (UHPFRC) beams under high strain rates is very limited (Fujikake et al. 2006, 2009). As, this is not always feasible to conduct large-scale test of UHPFRC beams under impact loading, a need for developing dependable 3D finite element model is now time worthy.

Numerical modeling of UHPFRC has been always challenging due to non-availability of post-peak behavior which may have a significant effect on the prediction of the damage parameters (Chen & Graybeal 2012). In this study, an attempt has been made to develop a finite element model that can be applied for a variety of UHPFRC beams subjected to impact loading. A parametric study has been also conducted to observe the influence of reinforcement ratio, drop height of impact load on the structural response of UHPFRC beams under low velocity impact loading.

2 FINITE ELEMENT MODELING AND VALIDATION

The numerical simulations were conducted using the ABAQUS code, which is a general FE analysis software for modeling the nonlinear mechanics of structures, fluids, and their interactions. To model the concrete material in ABAQUS, various models are available in the software library. The “concrete damaged plasticity” was chosen to model UHPC in this study. So far, no analytical model has been developed for predicting the compressive and tensile behavior of UHPC up to the knowledge of the authors. The compressive strength of UHPC is 152 MPa which is shown in Figure 1 and the tensile strength is taken as 6.40 MPa. Other input parameters are collected from literature (Chen & Graybeal 2012). Table 1 shows the input parameters used in the damage model.
Table 1. Ultra-High-Performance Concrete parameters used in the plastic damage model

<table>
<thead>
<tr>
<th>Concrete Strength (MPa)</th>
<th>Mass Density (ton/mm³)</th>
<th>Young’s Modulus (MPa)</th>
<th>Poisson’s Ratio</th>
<th>Dilation Angle ψ (Degrees)</th>
<th>Eccentricity fbo/co</th>
<th>b_c/b_t</th>
</tr>
</thead>
<tbody>
<tr>
<td>152</td>
<td>2.565E-009</td>
<td>44000</td>
<td>0.18</td>
<td>15</td>
<td>0.1</td>
<td>1.16</td>
</tr>
</tbody>
</table>

Reinforcing steel has been modeled using a 2-noded linear 3-D truss element (T3D2). Bilinear stress-strain curves are adopted for the simulation. The other parameters used to define the behavior of reinforcing steel are shown in Table 2.

Table 2. Parameters of reinforcing steel

<table>
<thead>
<tr>
<th>Type</th>
<th>Poisson’s ratio</th>
<th>Elastic Modulus (MPa)</th>
<th>Mass Density (ton/mm³)</th>
<th>Yield Stress (MPa)</th>
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<td>Steel</td>
<td>0.3</td>
<td>200,000</td>
<td>7.85E-009</td>
<td>517</td>
</tr>
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</table>

The proposed finite element model has been validated with two full-scale experimentally tested UHPFRC beams from the study “Response of ultra-high-performance fiber-reinforced concrete beams with continuous steel reinforcement subjected to low-velocity impact loading” by Yoo et al. (2015). The two specimens had two different reinforcement ratios. Figure 2 (a, b, & c) shows the experimental setup along with the cross sections.

Figure 2. (a) Experimental Program (b) cross section details of UH-N (c) UH-S-0.53% (Yoo et al. 2015)

After performing a mesh sensitivity analysis, 10 mm mesh size has been selected for the beam which can predict the experimental results quite satisfactorily in a reduced computational time. However, 20 mm mesh size is adopted for hammer. A single impact load was modeled to apply impact at the mid-span by dropping a free-falling weight of 270 kg, and the striking face consisted of a 20-mm thick and 40 x 210 mm sized rectangular steel plate with a flat contact surface.
just like the experimental setup. Fixed support conditions are applied to the top and bottom plate at both ends. The assigned impact velocity is 5.6 m/s which is similar to the experiment. The loading conditions and mesh configuration are presented in Figure 3 and Figure 4.

![Fixed Support](image1)

Figure 3. Load and Boundary condition

![Mesh Configuration](image2)

Figure 4. Mesh Configuration

Figures 5 and 6 show a comparison between the experimental results and finite element model to validate competency of FEM to foresee the maximum deflection, residual deflection and overall behavior of UHPFRC beams. It is evident from Figure 5 that the numerical model is capable of predicting the overall structural response. All test specimens exhibited a similar shape of the deflection–time history curves. The forced response shows a half sine wave followed by the residual deflection during the free vibration phase (after the removal of the drop weight effect). Both maximum deflection and residual deflection decrease with addition of reinforcement. The experimental ratio of maximum deflection to residual deflection for UH-N and UH-S-0.53% are found 2.20 and 2.39, respectively which are very close to the FEM (2.57 and 2.59 respectively). Figure 6 also confirms that FEM is in a very good agreement with test results. The results from experimental program and FEM are summarized in Table 3.

<table>
<thead>
<tr>
<th>Beam Designation</th>
<th>Reinforcement Ratio (ρ)</th>
<th>Maximum Deflection (mm)</th>
<th>Residual Deflection (mm)</th>
<th>Max./Res.</th>
</tr>
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<tbody>
<tr>
<td></td>
<td></td>
<td>Experiment</td>
<td>FEM</td>
<td>% error</td>
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<tr>
<td>UH-N</td>
<td>0</td>
<td>19.80</td>
<td>19.67</td>
<td>0.66</td>
</tr>
<tr>
<td>UH-S-0.53%</td>
<td>0.53%</td>
<td>16.65</td>
<td>16.98</td>
<td>1.98</td>
</tr>
</tbody>
</table>
Figure 5. Deflection-time history at midspan

Figure 6. Comparison of maximum and residual deflections between experimental program and FEM

The kinetic energy was reported 4.2 J (kg\(\cdot\)m\(^2\)/s\(^2\)) in the literature which is almost identical to the internal energy found in the FEM model. Figure 7 shows the internal energy-time history of UH-N obtained from FEM.

Figure 7. Internal energy-time history of whole beam

3 RESULTS AND DISCUSSION

The following section presents the impact response of UHPC beams under various parameters using the finite element code. The key parameters were the reinforcement ratio \(\rho\), and the drop height \(h\). The specimens were named as UH-\(\rho\)S-\(h\). D13 steel bar is used in each beam. The output parameters that had been extracted from the analysis are: the mid span deflection, residual deflection, and strain rate for both concrete and steel. The deflection-time history is generated from the numerical analyses for each parametric UHPFRC beam. The effects of the selected geometric
parameters on the enhancement of ultimate load carrying capacity and failure mode were also investigated in this study. The cross section of each UHPFRC beam is shown in Figure 8.

![Cross Sections of UHPFRC beams](image)

**Figure 8. Cross Sections of (a) UH-0S-50 (b) UH-0.53S-50 (c) UH-1.05S-50 (d) UH-1.71S-50 (e) UH-2.28S-50**

3.1 Effect of Reinforcement Ratio

Figure 9 (a) shows the numerically obtained deflection time history of UHPFRC beams under impact loading having five different reinforcement ratios (0, 0.53%, 1.05%, 1.71% and 2.28%). In general, it can be said that both the maximum deflection and residual deflection decrease with increasing reinforcement ratio.

![Deflection-time history](image)

**Figure 9. (a) Deflection-time history (b) Comparison of maximum and residual deflections according to reinforcement ratio**

Figure 9 (a) also shows that in the early stages of loading, the response curves follow a typical half sine curve followed by a plastic deformation till the failure strain. It can be observed that $\rho$ had a small effect on the initial slope of the deflection-time history. However, as the $\rho$ increases, the maximum deflection point moves to a lower level. Maximum deflection at mid span decreased by 37% when $\rho$ increased from 0% to 2.28%. In addition, the slope of the inelastic branch of the curves decrease with the increase of $\rho$.

Figure 9 (b) also confirms that the maximum deflection to residual deflection is increased by 3.4% with the increase in reinforcement ratio. This indicates that a higher reinforcement ratio provides better behavior in terms of deflection recovery.
3.2 Effect of Drop Height of Impact Load

The drop height of impact load is varied from 100 mm to 25 mm to observe the effect on maximum deflection at midspan. The impact velocity was kept constant for all the specimens. Figures 10 (a) and (b) depict that for a specific reinforcement ratio, the maximum deflection increases with decreasing drop height. Without reinforcement, the beams have experienced 16% more deflection when the drop height decreases to 25 mm, whereas the value is 14.7% when the reinforcement ratio is 1.71%.

![Figure 10](image_url)

Figure 10. Deflection-time history for different drop heights (a) \( \rho = 0\% \) (b) \( \rho = 1.71\% \)

3.3 Axial Strain Response under Impact Load

Figure 11 represents the axial strain time history of steel reinforcements for the simulated beams. It is observed from Figure 11 that with the increase of reinforcement ratio, the axial strain of reinforcement bars decreases extensively. However, most of the steel bars were not yielded except the beam having reinforcement ratio 0.53%.

![Figure 11](image_url)

Figure 11. Tensile Strain-time history

Figures 12 (a) and (b) represent the strain response of the UHPFRC beam having reinforcement ratios 0% and 2.28% respectively. It is evident from Figures 12 (a) and (b) that addition of reinforcement can significantly decrease the tensile strain of concrete at the bottom surface. The tensile strain is decreased by almost 50% when reinforcement ratio increased to 2.28%. In Figure 12 (a), red zone in the bottom surface of UH-0S-50 beam indicates the maximum tensile stain of 0.015 mm/mm which ensures the concrete failure. However, maximum strain of 0.006 is obtained for UH-2.28S-50 which is shown in the red zone of Figure 12 (b). The blue zone of Figure 12 (a) represents the compressive strain of 0.0018 mm/mm to 0.0035 mm/mm which almost disappears when \( \rho \) increases to 2.28%.
4 CONCLUSIONS

A commonly used software package ABAQUS was used for the simulations. The main aim of the finite element study is to develop a FE model which can simulate impact loading for different UHPFRC beams. The maximum measured displacement is found in mid span. The variation in reinforcement ratio has significant effect on the deflection recovery after impact. It is found that maximum deflection at mid span decreased by 37% when $\rho$ increased from 0% to 2.28%. In addition, the tensile strain in steel bars is decreased by almost 50% when reinforcement ratio increased from 0% to 2.28%. Also drop height has considerable effect on mid span deflection. To predict the impact response of UHPFRC beams more accurately further experimental works should be conducted.

REFERENCES


Seismic behavior of RC Beam-Column Connections Wrapped with Engineered Cementitious Composite Analyzed by Extended Finite Element Modeling (XFEM)

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**KEYWORDS:**

ECC, Crack, Beam-Column connection, Concrete, XFEM

**ABSTRACT:**

The applications of fiber reinforced concrete FRC to strengthen reinforced concrete (RC) structures has increased over last decades. Engineered cementitious composite (ECC) is a class of cement-based composite with fiber additive. The column-beam connection is one the critical regions when RC structures are subjected to cyclic loads. Special moment resisting frames (SMF) is one of the reinforced concrete alternatives for low and medium-rise building in hazardous seismic zones. Strengthening RC frames with ECC layer is an easy-to-apply method which provides superior mechanical properties due to ECC high tensile strength and prevents crack propagation. Concrete brittle failure is transformed to more ductile failure mechanism with adding ECC. This attribute of ECC is due to fibers energy dissipation in the complex loading during an earthquake. ECC prevent the reinforced-concrete columns from creating plastic hinges which is resulted in sudden collapse of RC structures. The objective of this numerical research is to analyze a two-story RC building with nonlinear extended finite element method (XFEM). The RC frame was wrapped with ECC layers which was reinforced with polyvinyl alcohol fibers. The results revealed the impact of one layer and two layers of ECC-wrap in the column-beam connections. In this analysis, the RC structure was subjected to an earthquake with magnitude of 6.5 Richter. The findings indicated the most damaged regions in RC structures are in the column-beam connection as well as the middle of the beam.
Seismic behavior of RC Beam-Column Connections Wrapped with Engineered Cementitious Composite Analyzed by Extended Finite Element Modeling (XFEM)

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ABSTRACT:
The applications of fiber reinforced concrete FRC to strengthen reinforced concrete (RC) structures has increased over last decades. Engineered cementitious composite (ECC) is a class of cement-based composite with fiber additive. The column-beam connection is one the critical regions when RC structures are subjected to cyclic loads. Special moment resisting frames (SMF) is one of the reinforced concrete alternatives for low and medium-rise building in hazardous seismic zones. Strengthening RC frames with ECC layer is an easy-to-apply method which provides superior mechanical properties due to ECC high tensile strength and prevents crack propagation. Concrete brittle failure is transformed to more ductile failure mechanism with adding ECC. This attribute of ECC is due to fibers energy dissipation in the complex loading during an earthquake. ECC prevent the reinforced-concrete columns from creating plastic hinges which is resulted in sudden collapse of RC structures. The objective of this numerical research is to analyze a two-story RC building with nonlinear extended finite element method (XFEM). The RC frame was wrapped with ECC layers which was reinforced with polyvinyl alcohol fibers. The results revealed the impact of one layer and two layers of ECC-wrap in the column-beam connections. In this analysis, the RC structure was subjected to an earthquake with magnitude of 6.5 Richter. The findings indicated the most damaged regions in RC structures are in the column-beam connection as well as the middle of the beam.

1 INTRODUCTION

FRC is widely used reinforcement for RC structures (A. Ghabarah, A. Said (2001) because of its light weight, corrosion resistance, outstanding strengthening properties and seismic performance. Reinforced concrete structures with FRC allows the engineers to provide repairing strategies as well as structural health monitoring (Baji, H. et al. 2015). Additionally, FRP has high Young’s modules of elasticity, and rupture strain that offers concrete higher strength and flexibility. Therefore, FRC layers strengthen the beam-to-column connections in RC structure (H.-K. Choi et al, 2013). Engineered cementitious composite (ECC) is a class of ultra-high FRC with polyvinyl alcohol fibers (S. Qudah and M. Maalej, 2014). However, other fibers can be fabricated to reinforce ECC (R. Zhang et al. 2015) and (J. Zhan & X. Ju, 2011). These fibers has outstanding engineering properties for the cyclic loads such as earthquake (N. Ganesan et al. 2014). The ECC improves brittle failure mechanism of concrete due to its better energy dissipation and more predictable failure mechanism (A. Spagnoli 2009). Additionally, ECC postpones progressive failure of concrete which is resulted in RC collapse (A. Khalili (2011).

Numerical analysis are categorized into extended finite element method (XFEM) and applied element method (AEM) (E. Mashaly et al. 2011), (H. W. Wanga et al. 2014) and (R. Kramer et al. 2013). After analysing the RC structure and predict the behaviour of RC structures subjected to seismic loads, rehabilitation strategies were applied with a layer of engineered cementitious composite. The variable is the thickness of the composite which was determined by trial and error method so that the composite layer prevented serious structural damage during seismic action (E. Giner et al. 2009) and (V. Gohel, 2013).

2 MODELING THE RC FRAME

In this computational analysis, a two-story building with the geometry that is shown in Figure 1 with XFEM and AEM was modeled and analyzed to predict the crack propagation behavior. The behavior of the RC beam-column connection subjected to gravity and seismic loads was assessed to evaluate the strain during an earthquake with the
magnitude of 6.5 Richter. The structure was simulated by Abacus-2D software with three methods including Contour Integral, XFEM, and virtual crack closure technique (VCCT). However, in Contour Integral, the crack path was not trackable and it could not be analyzed the structure subjected to dynamic loads. The XFEM and VCCT could be used in implicit dynamic cases.

![Figure 1. The geometry of RC structure.](image1)

Among all methods, XFEM and AEM are the best method for tracking concrete cracks path and the time of crack propagation occurrence. The spring element was modeled in Figure 2.

![Figure 2. The spring element in AEM.](image2)

In the AEM analysis, the beginning point of crack development, failure mechanism and collapse time was provided. The non-linear analysis was based on acceleration of 10 second in Koyanangar of India. The magnitude of earthquake was assumed 6.5 Richter. Every 0.02 second the output was recorded.

**3 MODELING WITH HYPERMESH SOFTWARE**

Hyper mesh model provided the complex model of 10 cm element size with boundary condition from empirical data (Figure 3). Finite element model with CPS4R was provided with a 4-degree freedom in X and Y direction and one rotation in Z direction which is considered as plane stress analysis. In Figure 4, the boundary condition and predicated crack areas can be seen.

![Figure 3. The Hyper Mesh Model.](image3)
In this model, 752 nodes and 564 square elements was analyzed. Aspect ratio of the dimension was 1.1 (the ideal aspect ratio is 1.0). Then the model in Hyper Mesh software was exported to Abaqus-2D.

4 MATERIALS PROPERTIES

Engineered cementitious composite (ECC) is more flexible than conventional concrete because it is reinforced with polymer fibres. The strain of polymer fibres is approximately 3-7% compared to cement which is 0.1%. Therefore, ECC is less brittle than cement. In this numerical analysis, one of the models was wrapped with one layer of ECC and the second model was wrapped with two layers of ECC.

Table 1. Concrete Mechanical Properties

<table>
<thead>
<tr>
<th>Concrete Properties</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s Modulus</td>
<td>31027 MPa</td>
</tr>
<tr>
<td>Poisson’s Ratio</td>
<td>0.15</td>
</tr>
<tr>
<td>Density</td>
<td>2643 Kg/m³</td>
</tr>
<tr>
<td>Dilation Angel</td>
<td>36.31</td>
</tr>
<tr>
<td>Compressive Initial Yield Stress</td>
<td>13.0 MPa</td>
</tr>
<tr>
<td>Compressive Ultimate Stress</td>
<td>24.1 MPa</td>
</tr>
<tr>
<td>Tensile Failure Stress</td>
<td>2.9 MPa</td>
</tr>
</tbody>
</table>

The properties of ECC layer is presented in Table 2. Cement Portland type 1 was used with coarse aggregate with maximum size of 19 mm. The sand was in compliance with ASTM C 778 for concrete 1 and 2. PVA fiber KURALON K-II REC 15 (length of 12 mm, diameter of 0.04 mm with elastic modulus of 37 GPa, tensile strength of 1600 MPa. For the first trial the thickness of ECC layer was chosen 2 mm for model 1 and two layers of the same ECC for model 2. The loading and boundary conditions were assumed the same. In the next step, the layer would increase if ECC layer did not meet the minimum requirement of strengthening for seismic load.

Table 2. Concrete and ECC Properties for model 1 and 2

<table>
<thead>
<tr>
<th>MATERIALS</th>
<th>F’C (MPA)</th>
<th>Ec (GPa)</th>
<th>EY (%)</th>
<th>C</th>
<th>S</th>
<th>CA</th>
<th>FA</th>
<th>W</th>
<th>SP</th>
<th>FIBER</th>
</tr>
</thead>
<tbody>
<tr>
<td>CONCRETE 1</td>
<td>52.3±3.6</td>
<td>28.6±1.8</td>
<td>0.01*</td>
<td>1</td>
<td>1.3</td>
<td>1.3</td>
<td>0</td>
<td>0.36</td>
<td>0.01</td>
<td>0</td>
</tr>
<tr>
<td>ECC 1</td>
<td>60.0±2.1</td>
<td>18.1±1.4</td>
<td>2.5</td>
<td>1</td>
<td>0.8</td>
<td>0</td>
<td>1.2</td>
<td>0.53</td>
<td>0.03</td>
<td>0.02</td>
</tr>
<tr>
<td>CONCRETE 2</td>
<td>45.6±1.0</td>
<td>-</td>
<td>0.01*</td>
<td>1</td>
<td>2.5</td>
<td>2.5</td>
<td>0</td>
<td>0.45</td>
<td>0.01</td>
<td>0</td>
</tr>
<tr>
<td>ECC 2</td>
<td>41.7±0.5</td>
<td>-</td>
<td>2.5</td>
<td>1</td>
<td>0.8</td>
<td>0</td>
<td>1.2</td>
<td>0.60</td>
<td>0.03</td>
<td>0.02</td>
</tr>
</tbody>
</table>
The element in the main model had three degree of freedom. Therefore, the spring degree of freedom was designed with 3 degree of freedom to meet mechanical properties of the RC frame (Figure 7). For the second floor beam also a small springer was assumed (Figure 8). Additionally, in the first model, a small spring was assumed between two frame sections as shown in Figure 8.
5 RESULTS AND VALIDATION OF AEM AND FEM

In two spans model, maximum strain occurred in the side columns, top part of the column-beam connections and bottom of the beams in the middle point as shown in Figure 9 (Huda Helmy et al, 2012). Figure 10 presents the result of current research which was validated by the results of Huda Helmy in 2012. In these figures, the variation of strain energy and plastic strain is presented. In the first model, the damaged region was column-beam connections as well as the column itself.

![Figure 9. Principle strain contours after internal column removal (Huda Helmy et al. 2012)](image)

![Figure 10. The result of the numerical analysis.](image)

![Figure 11. Energy density graph verses time in the end node of spring for model 2](image)

![Figure 12. Plastic strain graph in the end node of spring for model 2](image)
6 CONCLUSIONS

Application of engineering cement composite (ECC) which is combination of cement with various type of fibers indicated an alternative for rehabilitation of reinforced concrete structures. FRC superior properties made this class of cementitious composite behavior better than conventional concrete. In this numerical research, a two-story building was analyzed and assessed with AEM for the impact of one layer and two layers of ECC on RC behavior. The results indicated the curtail region in RC frame are in the column-beam connections as well as middle of the beam. Additionally, the variation of plastic strain, strain energy, and energy density were recorded. The magnitude of earthquake of 6.5 Richter was applied to the RC structure and the results for every two seconds were presented in Figure 11-14. The finding indicated two layers of ECC improved brittle failure mechanism of concrete due to its better energy dissipation and more predictable failure mechanism.

REFERENCES


**PAPER TITLE**
Effect of Dispersion and Quality Control of Multi-Walled Carbon Nanotubes on Cementitious Nanocomposite Subjected to Impact Load

<table>
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<tbody>
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**KEYWORDS:**
Nanocomposite; Carbon nanotubes; Cement, Dispersion; Strength

**ABSTRACT:**
For over fifty years, fibers of all types have been employed to augmented mechanical properties of cement-based composites. Conventional reinforced concrete evolution began with producing high-strength, high-performance, and ultra-high-performance concrete (HSC, HPC, and UHPC) by utilizing smaller particle sizes. However, nano-engineered concrete (NEC) is revolutionizing the world of concrete recently. Despite exceptionally high strength and promising mechanical properties that nanoparticles provides for cement-based composite, there are still two obstacles to reach maximum strength that nanomaterials such as carbon nanotubes (CNTs) potential offers for concrete reinforcement. CNT is a hydrophobic material with strong Van der Waals forces that leads to inadequate interfacial bond between CNTs and water as well as cement matrix. Therefore, homogeneous matrix production is extremely challenging. The contemporary methods to achieve high dispersed CNTs is to utilize the ultrasonic vibration energy [32] to split up bundles of carbon nanotubes. Additionally, surface-treated CNTs (functionalized) provide better interfacial bonding to reveal outstanding properties of carbon nanotubes. The aim of this research is to develop a dispersion method for cementitious composite incorporating multi-walled carbon nanotubes (MWNTs) to enhance dispersion quality. Another objective of this research is to investigate the best quality control methods to ensure sufficient uniform dispersion of MWNTs in cementitious matrix as well as sample preparation procedure for field emission scanning microscope (FESEM). The results indicated the effect of dispersion of MWNTs on impact characterization of cementitious nanocomposite.
Effect of Dispersion of Multi-Walled Carbon Nanotubes on the Impact Characterization of the Cementitious Nanocomposite

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1 INTRODUCTION

Concrete is widely used material in construction industry. Concrete types with ultra-high strength has received much interest among researcher for variety of applications. With advancement of technology in nanotechnology era, reinforcing cement-based composite with nanomaterials was an innovative method to enhance engineering properties of cement-based composites (Isfahani et al. 2016). Nanotubes with aspect ratio >>1, were considered one of the best reinforcement to compensate strength reduction due to using graphene and enhance mechanical properties. These properties made carbon nanotubes (CNTs) one of the best candidates for reinforcing cementitious composites (Nadi et al. 2016). Among all nanoparticles, multi-walled carbon nanotubes has outstanding mechanical properties such as Young’s modulus of 0.45 TPa and tensile strength of approximately 3.6 GPa (Xie et al. 2000 & Styonski 2015).

However, there are some drawback to fabricate CNTs into cementious materials. The CNTs tendency to bundle and adhere together is problematic because of Van der Waals forces. Second problem is CNTs are considered hydrophobic material while water is the main contributor of all types of cement-based composites. Thus, CNTs is not dispersed in water because CNTs are not capable of generating adequate interfacial bonds with cement matrices (Konsta-Gdoutos et al. 2010). Therefore, the challenging part of producing cementitious nanocomposite is dispersion of carbon nanotubes in water so that water and carbon atoms create a homogeneous liquid for cement matrix. The contemporary method to achieve high quality dispersion and deagglomeration includes ultrasonic technique (Konsta-Gdoutos et al. 2010 & Musso et al. 2009) and using functionalyzed CNTs (Li et al. 2005 & Muss et al. 2009). However, excessive use of ultrasonic energy may damage the carbon nanotubes structures and decrease CNTs properties (Rausch et al. 2010).

Many researchers used Carboxylate acid functionalized carbon nanotubes; various solvent; centrifuge stirring CNTs, and heating after sonication of CNT to increase dispersion effectiveness and enhance dispersion quality. In fact, covalent bond between CNTs sidewall and chemical functionalization increases bonding with CNTs and composite matrix. In this research both methods were utilized to provide the best outcome. Cementitious nanocomposite is an emerging field with limited laboratory research. Thus, dispersion of CNTs procedure has not developed yet. CNTs fabrication conducted with various method and sonication process (Li et al. 2013 & Bhari et al. 2014 & Wang et al 2013 & Jiang et al. 2003)

2 MATERIALS

The multi-walled carbon nanotubes (Figure 1) physical attributes that used including 10-20 µm length, 30-50 µm outer diameter, purity >95wt%, and specific surface area >60 m²/g. Multi-walled carbon nanotubes for Research
and Development (R&D) were used in this experiment to avoid tendency of entanglement in commercialized CNTs. The carbon nanotubes used in this research is COOH (carboxyl groups) functionalized to achieve better dispersion and longer stabilization of sonicated CNT.

![Image of MWCNTs](image1)

Figure 1. Transmission Electron Microscopy (TEM) image of MWCNTs with purity of greater than 95wt % with 30-50 nm OD (Source: Cheap Tubes)

Ordinary Type I Portland (ASTM C150) cement was used in this experiment. The cement was stored in the lab in air-tight plastic container to avoid hydration of cement in the lab. Type I cement contains 50% C3S, 24% C2S, 11% C3A and 8% C4AF. Water to cement ratio of 0.4 was added to make cement mortar. To assess the effect of MWCNTs, no additive and superplasticizer were added to the matrix.

3 TEST METHODOLOGY

3.1 DISPERSION PROCEDURE

In theory, uniform dispersion of MWCNTs is required to obtain high potential of carbon nanotubes. However, there is no standard procedure to disperse carbon nanotubes. Due to Van der Waals forces among CNTs mechanical agitation is needed for proper CNTs dispersion. Ultrasonic high-frequently sound waves were used with programmable QSONICA Q500 (Baig et al. 2018 & Gerges et al. 2015 & Wang et al. 2015) sonicator (Figure 2b). Sonication of CNTs overcome the bonding force and break down intermolecular bonds and allow water to react with CNTs.

3.2 PROGRAM SONICATOR

The sonicator was programed for 20 minutes and amplitude of 30% relatively. The total time of sonication was an hour (three 20-minute cycles). Due to high energy of the sonicator, the ice bath must be replaced regularly. An adjustable pulse time was programmed to be on for three seconds and off for two seconds to prevent excessive heat produce. An elapsed time indicator recorded the amount of time the dispersion process took to complete.

The chemical reaction of CNTs and water under ultrasonic mixture generates considerable heat energy. Therefore, an ice bath was required to prevent rapid evaporation of mixture during sonication process (Figure 3). According to the lab absorption, replacing the ice bath prevented considerable water evaporation. By weighting the CNT mix before and after the sonication process, it was found out that approximately 1.0% of water was reduced after an hour sonication. Therefore, in this experiment water-to-cement ratio was corrected based on the amount of water evaporation during sonication process.

![Image of sonication process](image2)

Figure 2. (a) Ice bath (b) Sonicators (QSONICA Q500) (c) Adjusting sonicator probe and (d) Dispersed multi-walled carbon nanotubes after an hour sonication at amplitude of 30%

3.3 MIXING CEMENT WITH DISPERSED CARBON NANOTUBES

Adding carbon nanotubes decreased the setting time of cementitious composite due to accelerating cement hydration. The experimental specimens was scaled down so mechanical mixture was not needed. Additionally, a standard mix procedure from ASTM and/or ACI has not provided a procedure for nanocomposite. Manual mixing of dispersed carbon nanotubes and cement was completed in 1-2 minutes due to quick hardening process of cementitious nanocomposite matrix. Additionally, control mix was produced without any CNTs for comparison purposes.
3.4 MOLDING PROCEDURE

According to ASTM C 109, after casting the cement mortar, the side of the cement mortar was tapped lightly to allow air bubbles on the cement mortar come to the surface and prevent further void in the hardened cement mortar. The impact mold size was cylinder with 50 mm diameter and 20 mm height. Cementitious nanocomposite matrices were placed into mold in two equal levels for impact specimen.

However, a lot of air bubbles was still existed in the mixture that caused lower strength in cementitious nanocomposite. To remove air bubbles within cement matrix, a small steel rods jabbed into each layer and mold were tapped 3-4 times after jabbing each layer. The top of cementitious nanocomposite cylinder was struck off with the rod. All molds were capped, marked and placed in ambient temperature until cementitious nanocomposite mix set up. After 24 hours, the specimens unmold according to ASTM C 192 (ASTM Standard C192/192M, 2002).

3.5 IMPACT TEST

In this experiment, American Concrete Institute (ACI) report (Measurement of Properties of Fiber Reinforced Concrete (Reapproved 2009)) was used, but the geometry was scaled down. The ACI sample geometry is recommended to be 150-mm diameter and a 50-mm height while the sample geometry scaled down to the diameter of 50 mm and height of 20 mm. The ACI procedure suggests a method to measure the mechanical properties of fiber reinforced concrete (FRC) under impact loading: ACI 544.2R-89 was initially developed for steel, glass, polymeric, and natural fibers cementitious composites (Moha et al. 2017). However, in absence of new report, the current ACI procedure was modified for CNTs reinforcement. In compliance with the ACI 544 committee report, the static impact energy on a disk specimen for a hammer of 620 gr weight from 100 mm height was employed.

5 QUALITY CONTROL

Improving mechanical engineering properties of cementitious nanocomposite is influenced by the quality of carbon nanotubes dispersion. The better the carbon nanotubes dispersed, the stronger the engineering properties of composite are. However, it is challenging to assess carbon nanotubes uniform dispersion. Therefore, the dispersion of CNTs in cementitious nanocomposite should be evaluated before producing specimen. In this research, to ensure minimum CNT agglomeration and maximum dispersion, two prototype samples were assessed by a non-destructive test.

The prototype sample made with cement, 0.2% multi-walled carbon nanotubes by weight of cement with water to a cement ratio of 0.4 (Table 1). According to ASTM C 109, after casting the cement mortar, the sides of the mold were tapped lightly to allow air bubbles to enter the cement mortar surface and avoid voiding in the hardened cement mortar. The specimen had been kept in oiled mold for 24 hours in moist conditions. Then the specimens were demolded and immersed in water until the day before the sample was dried and coated with a thin layer of gold. Field Emission Scanning Electron Microscope (FESEM) image test was implemented for 7 days.

Table 1. Mix Design for Quality Control Sample

<table>
<thead>
<tr>
<th>MATERIALS</th>
<th>WEIGHT</th>
<th>UNIT</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEMENT</td>
<td>112.38</td>
<td>gr</td>
</tr>
<tr>
<td>WATER</td>
<td>44.95</td>
<td>gr</td>
</tr>
<tr>
<td>MWCNTS (0.2% BY WEIGHT OF</td>
<td>0.22</td>
<td>gr</td>
</tr>
<tr>
<td>CEMENT)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

For the first prototype sample, the sonication process time was two 30-minute (total of 60 minutes) sessions with amplitude of 20% with one-minute break interval between the two sonications. In this mix, due to low w/c ratio, more water was added to the mixture to improve workability. MWCNTs cannot disperse in water without sonication. Therefore, the MWCNTs cannot be recognize in the FESEM images and cementitious crystals dominate the mixtures. Additionally, there is not sufficient bond with MWCNTs and cement crystals. In addition, coating with thin layer of gold was not applied for this sample.

After several mix design, the final sample mix design made of 0.2wt% multi-walled carbon nanotubes by weight of cement with water-to-cement ratio of 0.4 (Table 2).

Table 2. Final mix Design for Sample

<table>
<thead>
<tr>
<th>MATERIALS</th>
<th>WEIGHT</th>
<th>UNIT</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEMENT</td>
<td>54.98</td>
<td>gr</td>
</tr>
<tr>
<td>WATER</td>
<td>27.49</td>
<td>gr</td>
</tr>
<tr>
<td>MWCNTS (0.2%)</td>
<td>0.11</td>
<td>gr</td>
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6 RESULTS
Two classes of electron microscopes include Scanning Electron Microscope (SEM) and Field Emission Electron Microscope (FESEM). However, FESEM provides images with higher resolution and magnification especially for cementitious nanocomposite as CNTs are dominated by cement crystals. FESEM is composed of high-vacuum equipment that allows electrons to move through the specimen and provide high resolution pictures. FESEM indicates the morphology and crystallography of cement-based nanocomposite. In this research, the FESEM instrument was used instead of SEM due to high-magnification images that FESEM provides.

The results for the first prototype sample indicated CNT agglomeration due to uniform dispersion and poor connection between cement crystal and multi-walled carbon nanotubes (Figure 3). This picture is the first prototype sample with low workability. The additional water after sonication created nonhomogeneous matrix with inadequate bond between CNT and water. The continuous time of sonication also affected the mortar with lower water-to-cement ratio. Additionally, in this mix water-to-cement ratio was not adjusted and water evaporation was neglected.

![Figure 3](image1.png)

Figure 3. The CNT was not dispersed uniformly growing cement crystal and agglomerated MWCNTs

Morphological analysis throughout FESEM indicated uniform dispersion of MWCNTS in the final matrix and bonding between MWCNTs and cement crystals (Figure 4). The results of impact test on these specimens demonstrated the higher strength of the cement-based composite under statics loads compared to the specimens with CNT agglomeration and weak bond between CNTs and cement crystals.

![Figure 4](image2.png)

Figure 4. Percolation of dispersed MWCNTs within matrix and micro-crack were bridged by multi-walled carbon nanotubes/bundle of MWCNTs within the cementitious matrix.

Air void residing in cementitious nanocomposite is one of the challenges as carbon nanotubes were added to the cement matrix. A small amount of entrapped air void in cementitious nanocomposite substantially reduced the composite mechanical properties of hardened cementitious nanocomposite. Carbon nanotubes in multi-walled carbon nanotubes significantly added air void due to microstructural change in the cement matrix. Additionally, all classes of carbon nanotubes accelerate setting time of fresh cement paste which is created more micro and nano cracks. Therefore, the entrapped air voids created in fresh cement paste reduced the engineering properties of hardened cementitious nanocomposite. Lower crack propagation in the well-dispersed CNTs mixes postponed ultimate failure. More importantly, sudden concrete failure is one of the drawbacks of using concrete structures especially under impact load. However, with modifying the CNT dispersion method cement-based composite ductility and durability were enhanced. The feasibility of CNTs application in concrete reinforcement depends on directly to the quality of CNTs dispersion.
The mixing techniques of this research significantly decreased entrapped air void. Figure 5 shows the air void is on the order of a nanometer (129.60nm). Integrity of cementitious matrix offered higher strength for the nanocomposite sample. Another challenge was crack propagation across entrapped air voids as load was applied. With proper CNT dispersion, cracks and trapped air voids was interrupted the load transfer when cementitious nanocomposite is tested mechanically. The failure mechanism of cementitious nanocomposite was influenced by micro and nano voids. Cracks quickly propagated through the composite where these air bubbles occurred. To increase the crack propagation and produce homogeneous matrix the CNTs dispersion quality is the main contributor.

6 ACKNOWLEDGEMENT

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ASTM International, West Conshohocken, PA, U.S.A


**PAPER TITLE**: Pathology and forensic engineering applied over a trolley railroad bed using non-destructive testing

**TRACK**: Asset management

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**KEYWORDS:**
- Infrastructure pathology
- Ultrasound testing
- Rebound hammer testing
- Carbonation testing and remaining life
- Structural modeling.

**ABSTRACT:**
Unexpected pathologies that appeared on the trolley railroad bed in a southern city in Ecuador represent a main concern for the construction agency and for the supervisors, the aim of this work is to perform a comprehensive assessment of the railroad infrastructure in order to find the basis of the problem and thus, present solution alternatives. Forensic engineering via non-destructive test were performed in order to characterize the infrastructure and its stresses, as a second step, and using the create pathology database, structural modeling with different scenarios (even including the actual deterioration locations) were performed in order to understand the railroad bed behavior; finally, consequent solution alternatives were proposed.

Detailed testing including cracks survey, ultrasound measurements, rebound hammer testing, carbonation and remaining life, among other were performed; the structural modeling were executed with ANSYS Software. The results showed that most of the cracks were originated by the lack of steel reinforcement during either the design or the construction phase, also due to concrete retraction processes because of significant temperature gradients; furthermore, the infrastructure showed construction deficiencies and holes (possibly owing to the lack of vibration and bad curing). Additionally, it was found that the carbonation attack reaches the 70% of the concrete covering, among isolated corrosion and other pathologies.

To conclude, detailed solution alternatives and analysis were presented along with the conclusions.
Pathology and forensic engineering applied over a trolley railroad bed using non-destructive testing

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1 INTRODUCTION

In past years traffic growth, around 5.8 percent annually according to the Mobility Municipal Agency (EMOV by its initials in Spanish), has been a significant problem for Cuenca, because the increment in congestion, noise, and pollution.

This is the main factor why the city hall implemented the Project of building and operating a trolley system in the city, the Project must comply with four main objectives: efficiency, connectivity, coverage, and decongestion.

Nevertheless, a significant number of cracks appeared over the trolley railroad bed along the whole project, which means a structural risk for the project.

OBJECTIVE: In order to know what type of problem, its severity and to be able to give a comprehensive solution, these work assess this problem by implementing a defined methodology including non-destructive testing. Among the most significant tests used are visual observation, ultrasound testing, carbonation, and sclerometric measurements. All of this in order to be able to build a sufficient database about the pathologies, and also in order to implement and run structural modeling of the trolley railroad bed. For the structural modeling, two scenarios were analyzed: railroad bed with, and without cracking.

Finally, possible deterioration causes were defined, and solution alternatives were proposed.

2 METHODOLOGY

A. Visual evaluation: pathologies analysis for concrete

Thirty-three slabs were analyzed for this project, schematic plots were built in order to find particular pathologies and also to find any patterns for the cracks, a photographic database also were constructed. Possible deterioration factors were found and will be explained below.

B. Ultrasound testing for base platform

For the ultrasonic testing, more emphasis were given for the railroad bed sectors were dynamic testing with the actual trolley trains are performed (as the most critical locations).

Quality criteria for concrete was analyzed as a function of the ultrasonic pulse speed following Agraval & Leslie & Cheesman theory and the American Society for Testing and Materials” (ASTM C 597).

C. Rebound hammer testing

Indirect compressive strength was analyzed according to the sclerometric test methodology in the same points were ultrasonic tests were performed; the final objective was to correlate both results and calibrate them with each other values. Furthermore, particular sclerometric curves, developed in previous studies, were used to avoid possible errors and improve precision in the compressive strength determinations. The ASTM C805 standard were followed strictly.
D. Carbonation, remaining life prediction

Phenolphthalein diluted at 1% in alcohol was used to determine values for the carbonation depth for all the concrete slabs conforming the railroad bed, carbonation testing were performed in the concrete that was defined as relatively old, in new concrete slabs the testing was not performed for obvious reasons. Test was performed in ten slabs in total, in two different positions, and the results were recorded in millimeters following the “Réunion Internationale des Laboratoires et Experts des Matériaux” (RILEM) method. Test on concrete with braces were chosen for building the remaining service life curves, and test on concrete without braces were chosen to correct the values found with the sclerometric testing.

E. Remaining service life

The maximum carbonation depth was considered for this analysis; carbonation speed, carbonation depth, and structure life are exponentially correlated as follows.

\[ X = K \times (t)^{0.5} \]  
(1)

Where:

- \( X \): carbonation depth [mm]
- \( K \): CO\(_2\) infiltration speed [mm/year\(^{0.5}\)]
- \( t \): time [years]

And for “t”:

\[ t = \left( \frac{X}{K} \right)^2 \]  
(2)

Finally, the remaining service life is calculated as follows:

\[ tr = t - (\Delta t) \]  
(3)

Where:

- \( tr \): remaining time. [years]
- \( \Delta t \): structure age (elapsed time from the year of construction to the year of evaluation). [years]

Following this method, \( k \) value varies from 2 to 15 mm/year\(^{0.5}\) depending of concrete quality.

Over the braces, a 100-mm. covering was considered, no covering was considered for locations without braces.

Notice that carbonations consequences on concrete with no steel is irrelevant.

Finally, the remaining service life was calculated with the following considerations:

- Highest Rate for CO\(_2\) infiltration (\( k = 3.50 \) mm/year\(^{0.5}\))
- Evaluation time \( t = 4 \) years
- Functional project service life: 30 years

F. Modeling

Structural modeling was performed with ANSYS software, the chosen most critical scenario, along with the base scenario, for the modeling were: no cracks on base structure railroad bed, and with cracks on the base structure. Furthermore, necessary inputs to reach a precise model were also considered, being: variations in bogies’ mass distribution, trolley car geometry, soil capacity, and concrete railroad bed characteristics.

Structural modeling Parameters

The trolley railroad bed was placed over a compacted soil with a strong capacity according to the public agency documents. For the modeling, it was considered that the soil do not have any deformation and that it is a rigid material.

The concrete bed has three different layers below the covering layer, that was not placed yet, therefore not considered in the modeling yet. These three layers (from top to bottom) are base, foundation, & cleaning; each one with its particular thickness and strength;

Another hypothesis for the modeling: the load configuration over the rails are uniform and half for each rail, since each steel rail is considered as a rigid material with no deformation,

For each side of the bi-block sleeper, the considered load is the half of each bogie axis, since the bogie have two wheels each side.
Seven bi-block sleepers along the bed are considered in the model due to the length of the trolley car considered. The dimensions for the bed considered in the structural modeling therefore is: 6.2 m by 2.3 m. each block of the sleeper is 64 cm. by 29 cm, separated 1.43 m Fig. 1

Considering all elements’ dimensions and separations, the figure below shows the hypothesis for the load considerations:

The type of structural modeling is linear static; with static loads multiplied by a dynamic coefficient of 1.5, therefore:

\[ \text{Load} = \text{dynamic load} = \text{dynamic doef.} \times \text{static load} \]

Table 1 below shows the considered load for the modeling:

<table>
<thead>
<tr>
<th>Condition</th>
<th>Static Q (KN)</th>
<th>Dynamic Q (KN)</th>
<th>1/2 Dynamic Q (KN)</th>
<th>50% Blocks under the wheel</th>
<th>25% Blocks adjacent to the wheel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Empty trolley car +3%</td>
<td>72.87</td>
<td>109.30</td>
<td>54.65</td>
<td>27.32</td>
<td>13.66</td>
</tr>
</tbody>
</table>

Recalling that on each modeled trolley railroad bed exists seven bi-block sleepers, the following structural modeling scenario is considered as the most critical Fig. 3:
When considering cracks, these pathologies are included in the modeling as very poor concrete with friction. In addition, the most critical scenario for cracks was considered, meaning all-through cracks. Fig. 4

3 TESTING RESULTS

It is imperative to know the actual causes for cracking in order to treat them correctly [1].

For each pathology, the most common treatments are proposed according to “Rehabilitación y mantenimiento de estructuras de concreto” [2].

A. Visual evaluation, concrete pathology analysis

It is noticeable that cracks over sleepers’ struts are due to an insufficient covering over them (5 – 10 mm). Fig. 5 shows these cracks and its treatment using epoxy-based products and asphalt-based products.

Besides these particular cracks, it was also noticeable the existence of other types of structural cracks that can present significant risk for the whole structure, crack located perpendicular to the rails with openings of around 7mm. the possible causes for them could be the lack of longitudinal reinforcement as shown in Fig. 6.
Fig. 6 perpendicular structural cracks below the rail

Fig. 7 cracks schematic configuration for slabs 1, 2 & 3

Fig. 7 shows the following: 1. cracks over sleepers’ struts are passive with openings around 3mm, for these cracks the recommendation is gravity cracks sealing injections. 2. Active cracks in-between the sleepers and below the rails, with openings around 7mm, for these cracks, elastomeric products are recommended.

B. Ultrasound tests over the railroad

Table 2 shows an extract from: “análisis de patologías en la plataforma de rieles del proyecto tranvía cuatro ríos de cuenca” [4].

<table>
<thead>
<tr>
<th>slab</th>
<th>Corrected speed (m/s)</th>
<th>Concrete condition (according to)</th>
<th>Leslie y Cheesman</th>
<th>Agraval</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1546.3</td>
<td>Very Poor</td>
<td>Poor</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2677.5</td>
<td>Poor</td>
<td>Regular</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>1706.95</td>
<td>Very Poor</td>
<td>Poor</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2341.5</td>
<td>Poor</td>
<td>Regular</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1613.85</td>
<td>Very Poor</td>
<td>Poor</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1661.8</td>
<td>Very Poor</td>
<td>Poor</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>1124.2</td>
<td>Very Poor</td>
<td>Poor</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2432.5</td>
<td>Poor</td>
<td>Regular</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>2394</td>
<td>Poor</td>
<td>Regular</td>
<td></td>
</tr>
<tr>
<td></td>
<td>7518</td>
<td>Excellent</td>
<td>Good</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 8 shows that, according to Leslie-Cheesman, only 3% of the slabs are in excellent condition, 1.5% of slabs in good condition, and, on the other hand, 26% are in poor condition, and an astonishing 70% of the slabs are in a very poor condition.

With the same trend, Fig. 9 shows that, according to Agraval, only 5% of the slabs are in good condition, and, on the other hand, 21% are in regular condition, and a surprising 74% of the slabs are in poor condition.
C. Rebound hammer test

Table 3 shows an extract of the results from the sclerometric tests and its relative compressive strength values, notice that these compressive strength results are already corrected for carbonation. [4]

**Table 3 Sclerometric Results.**

<table>
<thead>
<tr>
<th>Slab</th>
<th>Rebound hammer index</th>
<th>Compressive Strength (kg/cm²)</th>
<th>Slab</th>
<th>Rebound hammer index</th>
<th>Compressive Strength (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>49,5</td>
<td>422</td>
<td>18</td>
<td>50</td>
<td>426</td>
</tr>
<tr>
<td></td>
<td>49,3</td>
<td>420</td>
<td></td>
<td>48</td>
<td>409</td>
</tr>
<tr>
<td>2</td>
<td>48,4</td>
<td>413</td>
<td>19</td>
<td>48,4</td>
<td>413</td>
</tr>
<tr>
<td></td>
<td>51,7</td>
<td>441</td>
<td></td>
<td>48,6</td>
<td>414</td>
</tr>
<tr>
<td>3</td>
<td>52,3</td>
<td>446</td>
<td>20</td>
<td>49,7</td>
<td>424</td>
</tr>
<tr>
<td></td>
<td>49,9</td>
<td>425</td>
<td></td>
<td>47,3</td>
<td>403</td>
</tr>
<tr>
<td>4</td>
<td>52</td>
<td>443</td>
<td>21</td>
<td>47,6</td>
<td>406</td>
</tr>
<tr>
<td></td>
<td>51,9</td>
<td>442</td>
<td></td>
<td>48,4</td>
<td>413</td>
</tr>
<tr>
<td>5</td>
<td>49,7</td>
<td>424</td>
<td>22</td>
<td>48,1</td>
<td>410</td>
</tr>
<tr>
<td></td>
<td>49,6</td>
<td>423</td>
<td></td>
<td>51,3</td>
<td>437</td>
</tr>
<tr>
<td>6</td>
<td>49,5</td>
<td>422</td>
<td>23</td>
<td>50,5</td>
<td>431</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>426</td>
<td></td>
<td>47,1</td>
<td>402</td>
</tr>
<tr>
<td>7</td>
<td>49,8</td>
<td>425</td>
<td>24</td>
<td>47,4</td>
<td>404</td>
</tr>
<tr>
<td></td>
<td>50,2</td>
<td>428</td>
<td></td>
<td>46,5</td>
<td>396</td>
</tr>
<tr>
<td>8</td>
<td>48</td>
<td>409</td>
<td>25</td>
<td>42,9</td>
<td>366</td>
</tr>
<tr>
<td></td>
<td>49,8</td>
<td>425</td>
<td></td>
<td>41,8</td>
<td>356</td>
</tr>
</tbody>
</table>
Even though this sclerometric results shows relatively good strengths, it is important to notice than this test has an important margin for error and it must be taken into consideration only as a preliminary diagnosis. Core extractions and compressive tests on them are recommended in order to calibrate the sclerometric results.

D. Carbonation tests

Table 4 & Table 5 shows the results for carbonation testing; carbonation depth, and its infiltration progress percentage. These values were used to calculate the correction factors for the remaining service life definition.

<table>
<thead>
<tr>
<th>Slab</th>
<th>Location</th>
<th>Depth (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>Between Railroad Sleeper 2 &amp; 3</td>
<td>min = 5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>max = 8</td>
</tr>
<tr>
<td>4</td>
<td>Between Railroad Sleeper 3 &amp; 4</td>
<td>min = 3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>max = 5</td>
</tr>
<tr>
<td>5</td>
<td>Between Railroad Sleeper 2 &amp; 3</td>
<td>min = 4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>max = 6</td>
</tr>
<tr>
<td>8</td>
<td>Between Railroad Sleeper 3 &amp; 4</td>
<td>min = 3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>max = 6</td>
</tr>
<tr>
<td>10</td>
<td>Between Railroad Sleeper 2 &amp; 3</td>
<td>min = 4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>max = 6</td>
</tr>
<tr>
<td>12</td>
<td>Between Railroad Sleeper 5 &amp; 6</td>
<td>min = 4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>max = 7</td>
</tr>
<tr>
<td>14</td>
<td>Between Railroad Sleeper 6 &amp; 7</td>
<td>min = 4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>max = 5</td>
</tr>
<tr>
<td>16</td>
<td>Between Railroad Sleeper 5 &amp; 6</td>
<td>min = 3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>max = 6</td>
</tr>
<tr>
<td>18</td>
<td>Between Railroad Sleeper 5 &amp; 6</td>
<td>min = 3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>max = 6</td>
</tr>
<tr>
<td>20</td>
<td>Between Railroad Sleeper 5 &amp; 6</td>
<td>min = 3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>max = 5</td>
</tr>
</tbody>
</table>

**Table 4 Carbonation depth**
Table 5 Carbonation depth for remaining service life definition

<table>
<thead>
<tr>
<th>Slab</th>
<th>Depth (mm)</th>
<th>Covering</th>
<th>Carbonation</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>min = 4</td>
<td>10</td>
<td>Reaches 60% of covering</td>
</tr>
<tr>
<td></td>
<td>max = 6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>min = 4</td>
<td>10</td>
<td>Reaches 70% of covering</td>
</tr>
<tr>
<td></td>
<td>max = 7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>min = 5</td>
<td>10</td>
<td>Reaches 70% of covering</td>
</tr>
<tr>
<td></td>
<td>max = 7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>min = 3</td>
<td>10</td>
<td>Reaches 60% of covering</td>
</tr>
<tr>
<td></td>
<td>max = 6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>min = 4</td>
<td>10</td>
<td>Reaches 60% of covering</td>
</tr>
<tr>
<td></td>
<td>max = 6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>min = 3</td>
<td>10</td>
<td>Reaches 50% of covering</td>
</tr>
<tr>
<td></td>
<td>max = 5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>min = 2</td>
<td>10</td>
<td>Reaches 50% of covering</td>
</tr>
<tr>
<td></td>
<td>max = 5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>min = 4</td>
<td>10</td>
<td>Reaches 70% of covering</td>
</tr>
<tr>
<td></td>
<td>max = 7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>min = 3</td>
<td>10</td>
<td>Reaches 50% of covering</td>
</tr>
<tr>
<td></td>
<td>max = 5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>min = 5</td>
<td>10</td>
<td>Reaches 70% of covering</td>
</tr>
<tr>
<td></td>
<td>max = 7</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

E. Remaining service life
Following equations (1), (2) y (3), remaining service life were calculated for the analyzed slabs, Table 6

Table 6 remaining service life

<table>
<thead>
<tr>
<th>Slab</th>
<th>Construction year</th>
<th>Evaluation year</th>
<th>Max. Carbonation depth (mm)</th>
<th>K, infiltration rate (mm/year)^0.5</th>
<th>Covering (mm)</th>
<th>Time: t=(X/K)^2 (years)</th>
<th>Remaining life, tr (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>2013</td>
<td>2017</td>
<td>6</td>
<td>3.00</td>
<td>10</td>
<td>11.1</td>
<td>7.1</td>
</tr>
<tr>
<td>4</td>
<td>2013</td>
<td>2017</td>
<td>7</td>
<td>3.50</td>
<td>10</td>
<td>8.2</td>
<td>4.2</td>
</tr>
<tr>
<td>5</td>
<td>2013</td>
<td>2017</td>
<td>7</td>
<td>3.50</td>
<td>10</td>
<td>8.2</td>
<td>4.2</td>
</tr>
<tr>
<td>8</td>
<td>2013</td>
<td>2017</td>
<td>6</td>
<td>3.00</td>
<td>10</td>
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<tr>
<td>10</td>
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<td>2017</td>
<td>6</td>
<td>3.00</td>
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<td>12</td>
<td>2013</td>
<td>2017</td>
<td>5</td>
<td>2.50</td>
<td>10</td>
<td>16.0</td>
<td>12.0</td>
</tr>
</tbody>
</table>
Using the root square model for time, the remaining life were calculated (Fig. 10) in this curve, it is observed that for 2021 the carbonation will reach the 10 mm covering. Giving a remaining service life of only 4 years.

F. Structural Modeling

The analyzed scenarios were modeled in ANSYS and are shown in figures below, Fig. 11 y Fig. 12 shows the scenarios were the loads are placed on the edges, Fig. 13 y Fig. 14 shows the models when loads are placed in the middle. Table 7 for loads at the edge, Table 8 for loads at the middle.
Table 7: Railroad Bed under loads at the edge

<table>
<thead>
<tr>
<th></th>
<th>Without cracks influence</th>
<th>With cracks influence</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum tensile stress (Pa)</td>
<td>33 076</td>
<td>164 720</td>
</tr>
<tr>
<td></td>
<td>– 63 034</td>
<td>-172 610</td>
</tr>
<tr>
<td>Maximum shear stress (Pa)</td>
<td>255 000</td>
<td>288 000</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Total strain (mm)</td>
<td>1.47 E-3</td>
<td>1.79 E-3</td>
</tr>
</tbody>
</table>

Fig. 12 Modeling for railroad bed with cracks, loads at the edge

Fig. 13 Modeling for railroad bed with no cracks, loads at the middle
for both load configurations (in the middle and at the edges) it is noticeable that the maximum tensile stress is five times more in the cases when there is the influences of the crack deterioration, when compared with the base scenario (no cracks influence). The shear stress also is bigger when there is cracks, but the difference in not as remarkable.

4 CONCLUSIONS

The cracks found in the slabs, that are located right on top of the bi-block sleepers are due to an insufficient cover over the braces. The categorization: passive cracks

The cracks found in the slabs, that are located in-between the bi-block sleepers are due to the lack of longitudinal reinforcement plus contractions for diary temperature gradients. The categorization: active cracks.

These cracks’ patterns are repetitive along the whole projects, and according to the structural modeling the rate of deterioration is exponentially worsen.

From the ultrasonic testing, where most of the analyzed concrete slabs were categorized as very poor or poor, conclusions about the quality of the concrete can be drawn, it is possible than all concrete have porous layers, indicating constructive inefficiencies like: poor vibration , lack of curing, excess of workability (water quantity), etcetera.

No conclusive analysis can be made from the sclerometric testing since it was not possible to extract cores for calibrations due to administrative prohibitions during the forensic engineering work.

Corrosion risk in imminent for the steel components of the sleepers like the braces and longitudinal reinforcement of the slaps, due to the insufficient covering, noticeable carbonation depth, and cracks right on top of these elements.
For the remaining service life analysis, conclusions show that CO2 infiltration reaches the 70% of the covering, giving a residual life of only 4 years before active corrosion starts.

From the structural model, results show that the railroad bed has a good behavior, for any load configuration, when there is no deterioration (presence of cracks). However, the structure do not behave correctly, when the presence of cracks are included in the model, having a significant increase (around five times) in the maximum tensile stresses and in the shear stresses.

Finally it is noticeable that cracks changes significantly the structural behavior of the trolley railroad bed, there is no monolithic performance, the stress distribution is changed drastically having stress and strains concentrations that will make the railroad bed to perform deficiently.

5 RECOMENDATIONS

It is recommended for the active cracks a sealant process using elastomeric or elastic type of products, so it will not fail when the cracks are working on the day-by-day temperature gradients.

Micro cement treatment or epoxy-based injections are recommended for passive cracks on top of the braces.

A comprehensive and detailed quality control is necessary for the following concrete foundations, so an adequate concrete strength can be guaranteed.

Increase concrete covering aver the braces can help avoiding stress concentrations and future passive cracks.

Core extractions are highly recommended along with sclerometric testing; by doing so, correct calibrations and correlations can be developed, therefore the precision can be significantly improved for the concrete strength prediction using rebound hammer testing.

REFERENCES


**PAPER TITLE**

Use of 3D Laser Triangulation Technology for Automated Inspection of Tunnels

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**KEYWORDS:**

3D Scanning, Tunnel, Automated Inspection

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**ABSTRACT:**

Significant advances in high-speed 3D imaging technology have been made in the last decade and there are now commercial, off-the-shelf, solutions for automatically evaluating infrastructure condition at high-speed. One such device is the Laser Tunnel Scanning System developed by Pavemetrics; a “spin-off” of the National Optics Institute of Canada.

The LTSS system utilizes laser line projectors, high-speed cameras and custom optics to acquire both 2D images, and high-resolution 3D profiles, of infrastructure surfaces at inspection vehicle speeds up to 180 km/h. 3D scanners of this sort offer numerous advantages to the traditional manual inspection including:

- Improved safety to staff
- Day-time or night-time operation
- Dramatically reduced inspection times
- Improved accuracy and reliability of results
- Creation of a permanent record of the tunnels appearance, condition and dimensions

This paper will explore the application of 3D laser scanning technology to the activity of tunnel inspection. The discussion will include a comparison to other technologies, such as LiDAR, as well as a discussion of the practical achievable levels of data precision and accuracy obtained during testing a number of large tunnels from around the world.
Use of 3D Laser Triangulation Technology for Automated Inspection of Tunnels

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INTRODUCTION

The LTSS is a 3D laser-based infrastructure assessment technology that has been continuously developed over the last 10 years by Pavemetrics (a commercial “spin-off” of Canada’s National Optics Institute) in close collaboration with the Ministry of Transportation of Quebec and numerous Infrastructure Management Consultants and other Departments of Transportation around the world.

The LTSS consists of laser line projectors, high-speed cameras and custom optics to acquire both 2D images and high-resolution 3D profiles of infrastructure surfaces at speeds up to 180 km/h. High resolution 2D and 3D data acquired are then processed using algorithms that automatically extract feature and defect information such as: cracking in pavements, missing fasteners in railways, rubber deposits on runways and chipping and cracking in tunnel linings.

This paper describes the function of this technology and its application to the inspection of tunnel infrastructure.

1. HARDWARE CONFIGURATION AND FUNCTION

The LTSS consists of two (2) or more laser scanners, a system controller, two (2) or more PCI express frame-grabbers, an industrial PC and an optical encoder to trigger data capture as the inspection vehicle traverses the length of the tunnel (Figure 1).

Before a tunnel inspection is performed the LTSS sensors and associated hardware are installed into an inspection vehicle which can be driven through the length of the tunnel. While a special vehicle type is not a
requirement of the system, a custom sensor mounting structure must be developed to properly position the sensors vis-à-vis the tunnel wall surface to a measurement distance of approximately 1.5-3.0m (Figure 2 and Figure 3).

Figure 2. LTSS Sensor Mounting (Example 1)

Figure 3. LTSS Sensor Mounting (Example 2)
1.1 PRINCIPLE OF OPERATION

The LTSS laser scanners operate on the principle of laser triangulation, with a high-speed near infrared laser projecting a line onto the tunnel lining surface and synchronized imaging sensor (an industrial camera) receiving the projected light (Figure 4). A triangle of known dimensions and angles is formed between the laser projector, camera and the projected laser line with the projected line being the top of the triangle and the project and imaging sensor forming the base.

Light Detection and Ranging (LiDAR) sensors on the other hand operate on the principle of Time of Flight with 3D dimensions being determined through the measurement of time required to send light to a surface and to receive its reflection. LiDAR sensors are well suited to applications requiring measurements over long distances at relatively lower resolutions and accuracies. Laser Triangulation on the other hand is ideally suited to high-detail inspections over short distances (Figure 5).

Figure 4. Laser Projector Operation

Figure 5. LiDAR Versus Laser Triangulation
Special filters are applied to the imaging sensor which make it more sensitive to the specific wavelength of the laser project while being less sensitive to other wavelengths present in natural sunlight as well as artificial lighting.

As the driving wheel of the inspection vehicle rotates, the coupled optical encoder generates electrical pulses which are fed to the system controller. These pulses inform the laser projectors and synchronized imaging sensors when to activate thus a laser line is projected for each encoder pulse (Figure 6). The resulting longitudinal scan interval can be as frequent as every 1 mm of travel down the tunnel at speeds up to 100 km/h. Less frequent intervals can be utilized in cases where higher inspection speeds are required (e.g., 2 mm at 200 km/h) or when the operator wishes to reduce data storage.

Transversal resolution of the sensor is a function of sensor mounting offset from the tunnel wall and is typically configured for 1 mm spacing such that the resulting tunnel scan is a 1 mm x 1 mm grid (Figure 7).

1.2 INTENSITY AND RANGE DATA

Laser light received by the imaging sensor is digitized by high-speed frame grabbers. The intensity of the laser light received is used to construct an “intensity image” of the tunnel wall’s surface while the specific position where the laser light is received in the imaging sensor translates to range (height) measurements for each intensity value thus producing a separate range image.

Intensity data is presented as a high-resolution, linescan-like image; wherein portions of the wall which are relatively speaking farther away from the sensor (e.g., concrete seams and hollow portions) are shown using white or light gray shades. Uniform surface areas of the tunnel wall are represented in darker shades of gray (Figure 8).
Intensity and range data are compressed in real-time using a proprietary data compression algorithm which reduces file size to just $\frac{1}{40}$th the size of the raw images. Images of individual scan lines from the length and right imaging sensors are automatically aligned and merged by the computer resulting in a continuous image of the surface being formed.

Integrated Inertial Measurement Units (IMUs) are used to track the changes in pitch, roll and heading for each sensor and processed by a special algorithm in order to remove their impact on scans before merging the scans from multiple sensors into a single surface which represents the true shape of the tunnel wall (Figure 9).

![Concrete seams](image1.png)
![A hollow portion of the wall](image2.png)

Figure 8. Intensity Image (left) and Range Image (right)

Figure 9. Scans from Multiple Sensors Merged into a Single Corrected 3D Surface

2. DATA ANALYSIS

One of the key benefits of this technology is the fact that data analysis can be performed through in the comfort of an office. This approach minimizes tunnel down-time as well as staff exposure to risk.

Inspections can be completed both manually, through intensity and range image review, as well as automatically through computer image processing. The high-resolution images produced by the system present a level of detail that approaches what can be seen in person when walking the tunnel (Figure 10).
Automated algorithms can be designed for analysis of intensity data, range data, inertial data as well as combinations of all three in order to detect and quantify defects which would otherwise be difficult to capture. Some of the defects, features and measurements which automated algorithms can detect and produce include:

- Feature detection
- Cracking
- Water ingress
- Chipping
- Mortar loss
- Joint measurement
- 3D surface modeling

2.1 FEATURE DETECTION

Often the first step in detecting defects of interest is to detect other features present, such as lighting fixtures and cabling, which the inspector wishes to ignore during their condition assessment (Figure 11, Figure 12, Figure 13). Of course, the inspector may also wish to specifically inventory the types and location of these features as well.
Figure 11. Feature Detection (Example 1)

Figure 12. Feature Detection (Example 2)
2.2 CRACK DETECTION

Once feature detection has been completed and the results excluded from the region of interest, crack detection can be performed without risk of false detection along the boundaries of features. Cracks at least 2 mm in width by 10 mm in length and 2 mm deep can be detected (Figure 14, Error! Reference source not found.). Crack location, length, width and depth are automatically assessed and a colour-coded map (showing severity) of their positions is created.
Figure 14. Crack Detection (Example 1)

Figure 15. Crack Detection (Example 2)
2.3 CHIP DETECTION

As is the case with crack detection, feature detection is first performed in order to omit the results from the region of interest. User-defined thresholds are used to target specific sizes of chips for detection in order to ignore very small superficial damage on the wall of the tunnel in favor of more significant defects of note. Chip location, length, width and depth are automatically determined (Figure 16, Figure 17 and Figure 18).

Figure 16. Chip Detection (Example 1)

Figure 17. Chip Detection (Example 2)
2.4 WATER INGRESS

Presence of water in a tunnel lining can indicate undetected cracking and other potentially significant surface damage. Presence of water can be detected through the analysis of intensity data specifically wherein wet surfaces present a much higher level of laser light reflection (Error! Reference source not found.). Location and surface area affected can be reported for water ingress.
2.5 MORTAR LOSS (BRICK TUNNELS)

The inspection of brick tunnels presents its own unique challenges compared to their more modern TBM brethren. Mortar loss is a defect which is particular to brick tunnels and can help detect dangerous loose bricks before they separate from the tunnel wall. Location and volume of mortar loss can be reported.

![Figure 20. Mortar loss detection](image)

3. REPORTING

A variety of output formats are available for results including JPEG images showing the tunnel wall as well as detected defects and features, 3D LAS files and XML files containing measurements (Figure 21 Error! Reference source not found.).
The use of XML, or “Extensible Markup Language,” for the output of measurements enables its use in virtually any reporting tool including simple spreadsheets, relational databases as well as custom tunnel management applications. XML reports typically include information such as date and time of detection, location of detection, and physical dimensions of each detected feature or defect (Figure 22).
4. SAMPLE PROJECT WORK

The LTSS was deployed by Euroconsult in the Spring of 2012 in order to scan the Guadarrama and Regajal tunnels in Spain.

The Guadarrama tunnel is the 5th longest tunnel in the world; 28,407m long western and 28,418m long eastern tubes. The Regajal tunnel is a shorter tunnel with a length of 2,200 m.

The entire length of both tunnels, as well as rail surfaces, were scanned using the LTSS, with 1 pass in each direction of each tunnel for a total of 4 passes at a speed of 20 km/h. Total scanning time for the Guadarrama tunnel was approximately 6 hours, and under 1 hour for the Regajal; thus minimizing tunnel downtown. 1mm resolution (longitudinally and transverse) 3D scans were collected for each tunnel with a vertical accuracy of 0.5mm.

Collected images were utilized to perform a detailed tunnel condition inspection and rail surface inspection from the safety and comfort of an office environment as opposed to in situ in the tunnel. Images were reviewed using a combination of automated algorithms and manual annotation of digital images in order to identify tunnel defects. A range of defects were identified and quantified from the digital scan, including: tunnel wall cracking, wet areas, overlapping of dowels as well as rail surface condition and rail fastener presence and condition.

Figure 23 presents an intensity data image of the Guadarrama tunnel where there were significant amounts of moisture are present on the walls (visible as darker areas in the intensity image). Automated algorithms can be used to detect moisture and present it as a highlighted overlay on top of intensity images for reporting purposes (Figure 24).
CONCLUSION
The LTSS is a versatile laser-based 3D scanning technology that can provide value to infrastructure managers across numerous modes of transportation.

Offering a much higher resolution and accuracy than typical LiDAR scanners, the LTSS is also capable of automated tunnel condition inspection. Automated algorithms can analyze intensity data, range data, inertial data as well as combinations of all three in order to detect and quantify defects which would otherwise be difficult to capture.

Defects, features and measurements which the LTSS can detect and produce include:

- Feature detection
- Cracking
- Water ingress
- Chipping
- Mortar loss
- Joint measurement
- 3D surface modelling

The key advantages to this technology are:

- Improved safety for staff due to reduced exposure time in the tunnel environment
- Reduced impact on tunnel operation due to reduced inspection times; day and night operation is possible and inspection speeds up to 180 km/h are supported
- Improved accuracy and reliability of results over visual manual inspections
- Creation of a permanent record of the tunnels appearance, condition and 3D shape
## New initiative to improve standards for tunnels

**ABSTRACT:**

The Norwegian Tunnel Safety Cluster (NTSC) have started a new initiative - The Norwegian Tunnel Safety Certification. This is an initiative that aims to improve standards for planning, building and operating tunnels, but also improve standards for structural elements and installations including infrastructure, ventilation, lighting, signs, safety equipment, control, monitoring and future Intelligent Transportation Systems (ITS).

Structures and installations today have weaknesses that affect reliability, availability, maintainability and safety in the tunnels. Tunnels with weaknesses affect costs, lifetime of construction and installations, and in many cases lead to high socio-economic costs in terms of more frequent closures and deviations. In addition, weaknesses could significantly affect safety. It is important to apply best practice, and the Norwegian Tunnels Safety Cluster aims to be in the lead for research, development and improvement in tunnel operations and safety.

The Building Information Model (BIM) standard, and BIM object libraries are an important prerequisite in the work, and the work includes to do the necessary steps to adapt each standard to BIM.

An important part of the work is to make sure the results are communicated and presented well in innovative ways, online, including in a Virtual Reality tunnel and presentation of technology in the new Norwegian Tunnel Safety Centre, planned to be established in Stavanger.

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**KEYWORDS:**

Tunnel construction, Tunnel installations, Tunnel Operations, Tunnel Safety, Tunnel Fires, RAMS analyses, BIM, Standardization
New initiative to improve standards for tunnels

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1 INTRODUCTION

Tunnel construction and installations have often major weaknesses that affect reliability, availability, maintainability and safety of tunnels. Tunnels with weaknesses largely affect costs, lifespan of construction and installations, and in many cases give high socio-economic costs in terms of more frequent shutdowns and less available road network. In addition, weaknesses could significantly affect safety.

It is important to apply new research, best practices and new technologies, and Norwegian Tunnel Safety Cluster will facilitate such activities to improve how we build and operate tunnels, and most important of all share results in innovative ways.

In this presentation and paper, the project manager will explore this topic in the context of how vulnerable tunnels are when it comes to safety.

After 7 years as National Tunnel Manager, following up the responsibility to the administrative authority in all phases for all tunnels in Norway, Arild Petter Sovik has taken the position as project manager for these projects in the Norwegian Tunnel Safety Cluster. We will introduce the needs of improved standards in the context of Norway’s 1180 tunnels, challenges in operating tunnels, and in the context of the 5 large recent tunnel fires in long single tube bidirectional tunnels. Fires that have shown that the preconditions for the self-rescue principle were absent as a result of the tunnel's safety equipment and emergency preparedness solution, resulting in several road-users being trapped in the smoke for up to 2 hours, which caused severe injuries to road-users.

2 WHY STANDARDISATION?

Standardisation is the process of implementing and developing technical standards based on a collaboration between organisations and companies including road/rail authorities, consultants, contractors, suppliers, and manufacturers, but also users and other national and international stakeholders are an important part of the collaboration.

In general, standardisation leads to better compatibility, interoperability, safety, repeatability and quality. Standardisation facilitates innovation because it provides structured methods that makes it easier to disseminate ideas and knowledge about leading technologies and best practices.

It is a common problem in the industry to “reinvent the wheel” every time a new tunnel is designed, and often experiences are not communicated to new projects. Function based standards will not only implement good procedures, and make leading technologies and best practices easier, but the establishment of committees will give an innovative and sustainable network for improvement and development of standards, and through this work contribute to improving tunnel operation and safety nationally and internationally.

3 TUNNELS – A VULNERABLE PART OF THE INFRASTRUCTURE

Risk is the probability of an event occurring and consequence of such an event. Probability is linked to, among other things, traffic volume, traffic congestion and special features and design of the road network.

Consequences can be significantly worse in the event of a road tunnel, and the characteristics of the tunnel will also affect this. Fire emergency preparedness in the area can also affect the consequences of fire in tunnels. Smoke is the main problem of a fire and the opportunity to reach another safe zone, which is not affected by smoke, is essential for the outcome of the incident.

Fire in a modern building will give those in the building a reasonable opportunity to escape to another safe zone in the presence of a fire. In a tunnel, you have to escape from your location to the portal outside of the tunnel. If
there is an escape tunnel, this can be used. For long one-tube road tunnels this may be several kilometers away. Often an unreasonable measure for safeguarding the principle of self-rescue.

Serious incidents that challenge the self-rescue principle is primarily incidents with large fire effects and cause a severe smoke situation, and incidents involving heavier vehicles. Leading experts are not too worried about fires in small vehicles. But still, a fire in a small vehicle can turn into a larger fire, involving several vehicles, and should be a part of the risk assessment.

Safety in tunnels is affected by many conditions. Both of technical, organizational and human nature, and the interaction between these. This applies to prevent incidents from occurring but also applies to prevent or minimize harmful effects of such incidents.

Knowledge of interaction between people, technology and organization is crucial for tunnel safety. Optimizing the interaction between people, technology and organization is important for a well-functioning and effective self-rescue. The interaction can be decisive when it comes to damage and the overall consequences of an incident.

The Traffic Control Center's operators and emergency services will, by implementing various measures and communicating with road users, play a key role in organizing and assisting in such a way that self-rescue is done as effectively as possible. Systems and routines must be simple and predictable for those involved. Simplicity and predictability reduce the likelihood of malfunctions. Coordination and interaction between the Traffic Control Centre operators and emergency services must be determined in advance as far as practicable, agreed on in contingency plan and regularly practiced. This is especially important for a Traffic Control Centre that have 250+ tunnels.

To maintain the safety level, it is important to have procedures, installations and equipment that make sure the self-evacuation strategies, rescue and firefighting efforts work as planned, and that installations and equipment are reliable, maintainable and available, and large incidents in Norway show that this is not always the case. We have experienced failures that significantly reduce the safety level. Without warning the Traffic Control Centre experiences failure in communication that affect the ventilation system during one of the most severe fires in Europe, contributing to a severe smoke situation, where 67 people are trapped in smoke for two hours.

4 THE INITIATIVE

Earlier this year, The Norwegian Tunnel Safety Cluster (NTSC) promoted its new initiative, The Norwegian Tunnel Safety Certification. The initiative that aims to improve standards for planning, building and operating tunnels, but also improve standards for structural elements and installations including infrastructure, ventilation, lighting, signs, safety equipment, control, monitoring and future Intelligent Transportation Systems.

The Building Information Model (BIM) standard, and BIM object libraries are an important prerequisite in the work, and the work includes to do the necessary steps to adapt each standard to BIM.

The work will be organized in several resource groups with participants from all across the industry, including participants from the authorities and other national and international stakeholders and organisations.

The resource groups will seek best practice, develop the standards, perform Reliability, Availability, Maintainability and Safety (RAMS) assessments and Health, Safety and Environment (HSE) assessments of the standards. Further on the standards will be subject of consultation, revisions and finally approved and implemented at authorities, organisations and in regulations:

1. **Seek practices/technology/research**
   Seeking best practice, new knowledge and new technology nationally and internationally

2. **Develop a standard**
   Create a preliminary specification of the standard, for assessments and further consultation with the industry.

3. **Assessments of standard**
   Conduct assessment of reliability, availability, maintainability and infrastructure safety (RAMS), Life Cycle Cost (LCC) and risk assessments, in addition to Health, Environment and Safety (HSE) Assessments.

4. **Consultation and adaptations**
   Consultation and adaptation process that include national and international stakeholders in the industry, and
authorities.

5. **Complete the standard, 3D and BIM**
   Complete the standard, with necessary adjustments to 3D and BIM, and facilitate approval and presentation in the Norwegian Tunnel Safety Center and in Virtual Reality (VR tunnel).

6. **Final approval and implementation**
   Approval and implementation by authorities, organizations, and other players in the industry.

   An important part of the work in the Norwegian Tunnel Safety Cluster is to make sure the results are communicated and presented well in the industry. The standard, and examples of use of the standards, will be available in innovative ways, online, including in a Virtual Reality tunnel and presentation of technology in the new Norwegian Tunnel Safety Centre, planned to be established in Stavanger.

   The first resource group started their work in June 2018 and the rest will start up consecutively in 2018. The resource groups are within the topics planning, analysis and execution (4 groups) and construction and installation (6 Groups):

   1. **Assessments in planning, design and construction**
      Reliability, Availability, Maintenance and Safety are ensured through assessments of RAMS, LCC, Risk and Emergency, and assessments must be ensured through well designed standardized methods and processes.

   2. **Tunnel management and preparedness planning**
      Owner’s responsibilities and other stakeholders’ responsibilities and interests must be safeguarded through well-defined and standardized routines, procedures, training, exercises and use of technology.

   3. **Operation and maintenance**
      Operation and maintenance of tunnels must be safeguarded through well-defined and standardized routines, procedures and use of technologies. Important to ensure reliability and functionality of important safety equipment.

   4. **BIM Standard and digital collaboration**
      Ensuring that BIM standard for tunnel construction, installations and equipment are implemented in accordance with best practices and standards.

   5. **Construction elements**
      Water and frost protection, barriers, road, side area, evacuation routes, evacuation shelters, drainage, pumps and basins.

   6. **Electrical infrastructure**
      Electrical infrastructure, emergency networks, radio coverage, mobile coverage and commercial networks.

   7. **Control, Monitoring and ITS**
      Control and monitoring systems and Intelligent Transport Systems (ITS).

   8. **Ventilation**
      Operating ventilation, fire ventilation, fans and ventilation towers.

   9. **Lighting, signs and markings**
      Tunnel lights, emergency lights, signs, markings, lead lines and moving barriers.

   10. **Safety Equipment**
      Camera surveillance, sensors / detectors, active fire protection, emergency cabinets, emergency telephones and voice notification.

   We have identified several success factors in the project that will be attractive for members and other stakeholders and make sure we will get the necessary participation and focus from the industry:

   - **Development based on knowledge and experience** in the whole industry (from owners to manufacturers)
   - **We focus on creating a team process that is effective**, and we will respect the members time and resources.
   - **The participants get the opportunity to influence** national and international development and improvement in their market.
• **The participants get the opportunity to grow their network** and develop products and services through collaboration and activities.

Well-defined goals that coincide with the goals of participating companies, national and international meeting arenas and presentation of results in innovative ways through the use of 3D models and online solutions are also success factors for this project.

**The Norwegian Tunnel Safety Centre**

Another relevant initiative starts at the same time, the establishment of a Norwegian Tunnel Safety Centre. The Norwegian Tunnel Safety Cluster establish facilities for research, development, demonstration and learning, with a main focus on training of tactics and execution in practice, for firefighters and incident command.

The tunnel facility will also be equipped with infrastructure and equipment specially designed for research and development and be geographically and academically affiliated with the university and a professional environment consisting of industry stakeholders and will be important in presenting new standards.

5 CONCLUSIONS

So far, the industry has responded well on this initiative and we see a large response in joining the project and in the individual resource groups. Work has already been done in identifying challenges, opportunities, strength and weaknesses in the industry, and new results will be presented and discussed in this presentation.
ABSTRACT:
Advanced satellite radar interferometry (InSAR) has been widely applied to civil engineering projects for monitoring ground deformation during all phases of a project, from design to construction and operation. The first reason for a wide application is that InSAR is a remote sensing technique and does not require the installation of ground-based instruments. By exploiting “natural targets” that already exist on the ground, a high density of measurement points is provided, which is not achievable with other traditional ground-based surveys. Secondly, taking advantage of historical satellite data archives acquired over the last two decades, InSAR can support the design phase of any infrastructure, by providing information about possible deformation phenomena already affecting the construction site. Finally, this technique is able to provide a synoptic view over an area much wider than one monitored with in situ instrumentations, detecting and monitoring unexpected deformation during the construction phase or residual deformation after completion, with a temporal frequency of a few days.
Examples of successful InSAR applications are illustrated to highlight the advantages of integrating InSAR data into an infrastructure monitoring plan in all stages of a construction project:

- Gran Paris Express (Paris): 15 years of monitoring more than 200 km of underground metro line and 68 new stations on a monthly basis during construction and after its completion.
- Canada Line Rapid Transit Metro (Vancouver): long-term monitoring project to detect surface deformation along the entire alignment prior to, during and after completion of construction.
- Variante di Valico (Italy): analysis of the relationship between tunneling and surface settlements along a slope under which large twin tunnels were bored in difficult geological and geotechnical conditions.

Finally, a near-real time monitoring service to support risk management along a linear infrastructure is discussed by illustrating the case of a 280 km-long highway in South Italy.
Satellite InSAR ground stability monitoring during the planning, construction and operation phases: highway and tunnel monitoring case studies

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1 INTRODUCTION

Monitoring surface deformation is fundamental for mitigating risks related to civil engineering projects during all phases of a project, from design to construction and operation. The instrumentation used for surface deformation monitoring in and around infrastructures is generally based on conventional survey techniques (total stations, levelling, GPS receivers, extensometers). However, none of these techniques offer the high density, bird’s eye view of the movement areas provided by satellite radar interferometry (InSAR).

The complementary use of space-based InSAR with traditional systems has been successfully applied in civil engineering projects worldwide, proving to be a strategic tool for detecting unexpected deformations and managing risks. Recently, InSAR has been also included in the “ITAtch Guidelines for Remote Measurements Monitoring Systems”, providing recommendations to assist tunnel designers, contractors and owners in understanding the benefits of including remote sensing techniques in their geotechnical monitoring plans.

After a brief introduction to InSAR techniques, this paper presents an overview of a number of applications of InSAR for roadway and tunneling projects.

Finally, thanks to the recent launch of the Sentinel-1 constellation of the European Space Agency, with a wide track coverage (250x150 km) and a short revisit time, a near-real time monitoring service for long linear infrastructures has been developed. The InSAR monitoring program for a 280 km-long highway in Southern Italy will be presented.

With continuous advances to the technology and new SAR satellite missions being launched by national space agencies and commercial operators yearly, InSAR has become a staple component of long-term geotechnical monitoring programs for civil engineering projects.

2 INSAR TECHNOLOGY

SAR Interferometry (InSAR) is a remote sensing technology that provides measurements of ground displacements (Gabriel et al., 1989; Massonnet and Feigl, 1998; Rosen et al., 2000; Bamler and Hartl, 1998). Radar sensors mounted on satellites acquire images of the Earth’s surface by emitting electromagnetic waves and analyzing the reflected signal. InSAR techniques consist of comparing the phase values of two SAR images, acquired at different times with similar looking angles. The phase difference is proportional to the target motion occurring along the sensor-target line-of-sight (LOS) direction during that time interval (Figure 1). As SAR satellites are continuously circumnavigating the globe, a number of radar images can be collected for the same area over time and information about the evolution of the earth’s surface can be extracted.

In the late 1990s, new Advanced DInSAR (A-DInSAR) techniques emerged in order to estimate and remove the atmospheric noises that affect basic DInSAR data and provide more accurate displacement measurements (sub-millimeter precision) by processing multiple images acquired over the same area over time. Permanent Scatterer Interferometry, the first A-DInSAR technique, identifies and monitors point-wise permanent scatterers (PS), pixels that display both stable amplitude and a coherent phase throughout every image of the dataset (Ferretti et al. 2000, 2001). PS are related to natural radar targets such as manmade structures (buildings, street lights, transmission towers, etc.) as well as rocky outcrops, un-vegetated Earth surfaces, boulders, and any linear structure that can reflect a signal back to the satellite. In order to detect the highest possible density of measurement points in non-urban areas, a new technique known as SqueeSAR™ was presented by Ferretti et al. in 2011, which extracts information from distributed scatterers (DS). This extends measurement point coverage to areas with limited infrastructure and light vegetation. Together, these two types of measurement points form a ground network of radar benchmarks, similar to a GPS (Global Positioning System) network and can be used for monitoring both the displacement of individual structures (a building, for instance), and the evolution of a large displacement field affecting hundreds of square kilometers.
The existence of low-resolution SAR data archives going back to the 1990s initially led to the extensive use of InSAR data to perform historical ground deformation analyses to assess any pre-existing ground deformation phenomenon prior to the design phase of a road or tunnel project. More recently, the use of high-resolution sensors has considerably increased the density of detectable information, up to thousands of measurement points per square kilometer in urban areas (Figure 2). In conjunction with this, the launch of satellites with a high frequency of acquisitions (up to a few days), combined with the development of sophisticated automatic processing algorithms made it possible to continuously provide reliable surface deformation measurements with each new satellite image.

Figure 1. An illustration showing the relationship between ground displacement and signal phase shift. This is the basic principle of InSAR for measuring ground movement.

Figure 2. From Bischoff et al. (2017): Low-resolution Sentinel-1 data (a) and high-resolution TerraSAR-X data (b) over East London, demonstrating agreement in regional displacement patterns despite the point density differences. Uplift in this area (blue) is linked to ground rebound as the water table rose after dewatering for Crossrail works was reduced.
Selected case studies will be presented in this section to illustrate the use of satellite radar data during various infrastructure project stages, from design to construction and operation.

The key characteristics of InSAR applications for civil engineering are:

- The wide coverage and high density of information provided by advanced InSAR techniques is not achievable with in-situ instrumentation. As InSAR is based on the use of natural radar targets that already exist over the ground, the technique provides thousands of measurement points per square km, extending the InSAR view well beyond the immediate surroundings of the alignment or construction site and making it possible to monitor wide infrastructure networks and their greater surroundings.
- Historical satellite data archives collected over the last two decades can be processed to detect pre-existing ground movement near the planned alignment or project site in order to estimate the impact of construction activities.
- The recent launch of satellites with high spatial resolution and high frequency of acquisitions, as well as the recent development of processing techniques provide near real-time and long-term stability monitoring solutions.

During the preliminary design phase of any infrastructure project, InSAR can provide historical ground motion information that contributes to the characterization of pre-existing deformation phenomena that can affect the future infrastructure. During the alignment planning stages of long, linear infrastructures, satellite remote-sensing data also offers the advantage of minimizing survey times and costs when covering wide areas.

During the construction phase, InSAR is integrated with localized in situ monitoring instrumentation to monitor a wide area for unexpected deformation phenomena that may be linked to the construction works.

Finally, during the operation of the infrastructure itself, most long-term monitoring strategies include periodical InSAR updates. Regular monitoring of infrastructures in operation supports the maintenance program, helping to identify possible structural weakness or damages and providing an early warning of possible accelerations in deformation affecting any structures.

3.1 Grand Paris Express (Paris, France)

Grand Paris Express is the largest transport development project in Europe. It consists of a fundamental redesign of the public transport network for the entire Paris metropolitan area, including 68 new stations and 200 kilometers of underground metro lines. Construction work began in mid-June 2016 and is intended to last almost until 2030. Because of the creation of numerous construction sites, most of them in urban areas and using complex underground drilling techniques, risk assessment and management plays a key role in both the project design and construction phases.

The Société du Grand Paris (SGP) is the public agency set up by the French government in 2010 to deliver the vision of Grand Paris Express. It leads all operations related to the construction of the new metro lines, stations, structures and facilities, acquisition of rolling stock for the infrastructure and development within and around the stations. From project conception to the execution of the Grand Paris Express, the SGP relies on specialized companies that support it in its role of project owner. In particular, GGP commissioned a historical SqueeSAR analysis to detect any preexisting movement affecting over the 200 km of the Grand Paris Express network alignment from April 1992 to March 2015 (Urdiroz et al. 2014).

The historical ground motion analysis included the processing of low-resolution satellite images (from the ERS and ENVISAT satellites of the European Space Agency), covering the period 1990-2010, and high-resolution satellite images (from COSMO-SkyMed of the Italian Space Agency and TerraSAR-X of the German Space Agency) over the more recent period 2011-2015 (Figure 3). This historical analysis has made it possible to create an extensive inventory of the ground surface behavior and the identification of vulnerable structures before the start of any construction work. In key areas where surface movements were detected, additional ground surveys have been or will be carried out if required to complement geological and geotechnical data.

As the first groundworks started in 2016, the TerraSAR-X high-resolution satellite has been used for detecting and measuring surface deformations related to the progression of the tunnel boring machine and complement the in-situ real time monitoring (precision levelling measurements, inclinometers, etc). Monthly SqueeSAR updates, based on a new satellite image every 11 days, provide an unrivalled measurement point density (> 10,000 points/km²) and are specifically designed to monitor the evolution of non-linear motion with a millimeter accuracy over the tunneling sites (Koudogbo et al. 2018).
3.2 Canada Line Rapid Transit Metro (Vancouver, Canada)

The Canada Line is the third rapid transit line built in the SkyTrain metro system in Metro Vancouver, British Columbia (Canada). It comprises 19.2 km of track from Vancouver to Richmond, with a 4 km spur line from Bridgeport Station, which connects to the airport (Figure 4a).

The construction started in 2005 and was completed in 2009. The line crossed at least two distinct types of terrain. The downtown core and most of the line north of the Fraser river lies on relatively competent glacial till while the southern sections of the line are sited on soft deltaic deposits of the Fraser river, mainly comprising silt and clay. The downtown section of the line was excavated with a TBM while between False Creek and the Fraser River a cut-and-cover method of excavation was used.

A long-term SqueeSAR monitoring program was established to detect surface deformation along the entire alignment prior to, during and after construction (Falorni & Iannacone, 2014). An extensive RADARSAT (of the Canadian Space Agency) satellite image archive covering the period 2001-2008 made it possible to detect the preexisting deformation and to monitor the displacement associated with construction of the tunnel (Figure 4).

Several areas of settlement in the soft sediments of the Fraser River along the southern portion of the Canada Line were detected but no large-scale deformation along the section of the line north of the Fraser River were observable. Figure 4a and 4b show the pre- and post-excavation displacement rates observed along a cut and cover section of the corridor. Measurement points on opposite sides of the excavation showed the likely presence of horizontal movement of buildings towards the excavation area, a clear signature corresponding to the timing of tunnel excavation.

Since the completion of construction in 2009, high-resolution COSMO-SkyMed (Italian Space Agency) imagery has been used to monitor the entire alignment.
3.3 Variante di Valico, A1 Highway (Northern Apennines, Italy)

The Variante di Valico is a deviation project for a portion of the Italian A1 highway in the central Apennines. Open to traffic since December 2015, the entire project spanned 62.5 km, of which 37 km involved adding a third lane on each side of the existing highway and 25 km included the construction of a new section, most of which consisted of viaducts and tunnels, the longest being 8.7 km in length. Now complete, the new section runs parallel to the old motorway and provides an alternative route in order to decrease traffic congestion.

Over one of the tunnel sites, deep-seated quiescent landslides were reactivated during the excavation (Barla et al. 2015). The twin three-lane tunnels, each with a cross-section area of 160 square meters, were excavated full-face by conventional methods. Systematic reinforcement measures by means of fiberglass dowels were applied and the final lining was always kept near the advancing tunnel face. Tunneling took place through a flysch rock mass, consisting of sandstone-mudstone layers with different thickness, with rock mass quality from fair to poor and, in some cases, very poor. The area above the tunnels was heavily monitored during the excavation using inclinometers and piezometers, including a number of robotic total stations for real time monitoring of villages. The two tunnels were excavated with one face preceding the other by 80-100 m, within controlled values of both the convergences of the tunnel perimeter and extrusion deformations ahead of the face.
With evidence of surface and subsurface movements occurring concurrently during tunnel excavation, the decision was taken to activate an InSAR monitoring plan. SAR imagery acquired by three different medium-resolution satellites over more than a decade (2003-2015) were processed with SqueeSAR to reconstruct a history of landslide behavior before the construction commenced (Figure 5).

Figure 5. Multi-temporal deformation maps over a section of tunnel. Each map represents the average yearly displacement rate for a specific period. On the right the displacement time series of some MP are reported.

A displacement rate of a few mm/year was observed before tunnel excavation, previous to the installation of any other conventional monitoring instrumentation. A sudden acceleration was observed during the excavation activity, starting from 2011 (displacement rate up to 60 mm/year between 2011 and 2013). Surface movement developed progressively and coincided with the tunnel excavation and face advancement, with clear evidence of reactivation of the deep-seated landslides.

The displacement time histories of some MPs are shown in Figure 6 and Figure 7, together with displacement data provided by robotic GPS stations. RADARSAT data (Figure 6) cover the 2003-2013 period and highlights the initial stage of the reactivation. Starting from April 2014, dual geometry TerraSAR-X images were processed, enabling the separation between vertical and E-W horizontal components of motion. A good fit between InSAR and GPS eastward and vertical components is shown in Figure 7.
After tunneling completion in November 2014, a progressive deceleration started to take place, even if at the end of March 2015, a complete stabilization was not yet reached over the entire area above the tunnels.

Starting from late 2014, a monitoring plan with high-resolution TerraSAR-X images was established. Monitoring will run until 2020 on a quarterly basis, in order to identify any possible correlation between the observed slope displacements and tunnel excavation, as well as to monitor the post-construction phase.

![InSAR and GPS displacement time-series. InSAR data are obtained by the processing of a descending RADARSAT imagery covering the period March 2003 – March 2013. GPS measurements started in January 2013. The comparison is performed by projecting GPS measurements along the satellite line of sight.](image)

![GPS and InSAR displacement time-series, vertical (blue) and E-W (red) components. InSAR data are obtained by the combination of ascending and descending TerraSAR-X datasets, covering the period April 2014 – March 2015.](image)

5 NEAR-REAL TIME INSAR MONITORING OVER LONG INFRASTRUCTURES

The case studies presented so far clearly highlight the advantages of integrating InSAR in all stages of a new infrastructure project, in combination with in situ, real-time monitoring techniques. While InSAR techniques have long been applied over both specific sites and large areas, results were not presented at a frequency to meet real time monitoring needs. The recent launch of short revisit time satellites has vastly improved data collection capacities and the development of processing techniques and tools that can support operational time schedules.

In particular, the Sentinel-1 constellation, launched in April 2014 within the ESA Copernicus program, consists of two satellites acquiring images over 6 days over Europe with a 250x150 km track coverage and free data access. By coupling the short revisiting time of Sentinel and other satellites, the wide-scale mapping capability, the regularity of acquisitions and the free data access with new automatic tools for data mining and screening of large datasets with millions of measurement points over thousands of square kilometers, regional continuous monitoring of ground deformation (data updated every new acquisition) is now being used operationally for numerous civil engineering projects. Such is the case in the region of Tuscany in Italy (Raspini et al. 2018).

Starting from this regional approach, a continuous monitoring service was developed along a 280km-long highway in Southern Italy. The monitoring plan consists of a multi-scale approach that helps optimize the timing and costs of mitigating risks and planning interventions. A first wide area level approach uses the Sentinel-1 capabilities to provide an update on the ground deformation over the entire highway with each new image acquisition. A specific data mining strategy was developed to highlight possible accelerations in motion affecting the highway itself or any deformation patterns near the route. By integrating this near-real time monitoring program with the preexisting in situ information, a map flagging any localized monitoring priorities is delivered every 12 days (Figure 8). This priority map classifies each highway sector by the urgency for a follow-up in-depth analysis. A Class 3 status indicates that no follow-up is needed as the area is stable. A Class 2 status triggers a first alert for a site-specific analysis, combining high-resolution satellite data with in situ surveys to better characterize the deformation phenomenon detected. A Class 1
classification generates an alert for an analysis over specific structures (Figure 9) and is activated in case the second level analysis confirms a higher risk.

Figure 8. Examples of priority maps obtained from the wide area analysis along the highway. The maps are updated with each new satellite acquisition.

Figure 9. Example of high resolution analysis activated over a specific viaduct that was flagged in the Class 2 analysis. The displacement time-series highlight an acceleration of the deformation in the last period.
6 CONCLUSIONS

Increasingly, space-based InSAR is being built into comprehensive geotechnical monitoring programs for large civil engineering projects as it offers a synoptic, wide-area view, which coupled with localized, traditional in-situ systems offers the most thorough combination of spatially and temporally dense ground deformation data with real-time updates of critical areas. Examples of InSAR for monitoring of three large civil engineering projects illustrate the applicability of this technique for providing valuable ground motion information during the project planning stages as well as during construction and once operation has begun. Finally, thanks to the recent launch of the Sentinel-1 constellation of the European Space Agency, with a wide track coverage (250x150 km) and a short revisit time, a near-real time monitoring service for long linear infrastructures has been developed. A long highway network in Southern Italy is being monitored using this type of alert service, which allows regional authorities and highway managers to pinpoint locations of high interest and focus resources for in-situ measurements where they are most needed.

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Unified Data Network For Tunnel Operations

ABSTRACT

At present, most tunnels have a fiber optic cable used for data transfer (video cameras, signaling, etc). For communication, leaky coaxial cables are run and VHF is the most used system. Hardly any tunnel has a tracking system and very few tunnels take full advantage of having a fiber optic infrastructure.

By installing Wireless Access Points (WAP) approximately every 500 meters, a full Wi-Fi coverage will be propagated throughout the tunnel. Having a Wi-Fi platform in the tunnel brings:

- Tracking by using RFID tags, knowing the precise location of people while underground
- Communications by Voice over Internet Protocol (VoIP) phones, radio or push-to-talk functions are also compatible.
- Wireless data transfer, providing access to the internet and allowing the use of smart phones.
- Tunnel access control.

For exceptional maintenance operations in locations where there is no infrastructure for communications or data transfer, Wireless Repeater Nodes (WRN) are a perfect fit. They are mobile self-meshing network extender/access points, intended for the extension of an existing Wi-Fi network. The WRN are designed to enable multiple nodes in a redundant mesh implementation, effectively filling communication black spots or “gaps”.

Having a unified communication and data network simplifies your tunnel operation and makes your system more reliable.

Guido Perez Manfredini
MST Global

INTRODUCTION

Currently, the construction sector is living in a new era where it is expected that lessons learned from the previous mistakes will allow more solid and robust activity. Now-a-days, the increasing pressure to reduce costs and increase profit margins remains. For this financial matter it is essential to get the most out of current assets; in other words, it is fundamental to improve productivity. The emergence of the industry 4.0 in all sectors, in a manner that has never been seen before, has and will have an immeasurable effect on all activities of the tunneling sector, and communications are not exempt.

Digital transformation is primarily a business transformation. People, not technology, are the most key component of this transformation. It affects all levels of an organization; therefore, leaders must ensure that this technology-based change can guide productivity gains and significant benefits. The digital transformation of construction will unlock unprecedented amounts of money. It is also relevant to mention that the transformation entails not only technology but equally, issues associated with culture, law, global and virtual teams, employment and new jobs, change and risk management, and many other subjects. This transformation requires a totally different view than those of the recent past.

Digital technologies are causing a high rate of destruction and creation of realities in different fields and industries, including construction. Innovation in communications, remote control, tracking, and data transfer are increasingly being applied to construction operations in the United States.

To innovate is to create new high-value solutions that solve problems that have not yet been resolved or that are poorly resolved. In construction, this translates into higher levels of productivity and operating margins, more production volume, job security, and fewer negative environmental impacts. Innovation generates
changes which involve uncertainties, and people’s natural reaction is to fear the unknown. The unknown being the fear of losing power and unemployment. The voices that announce robotization and artificial intelligence will cause widespread unemployment, are on the rise.

Construction leaders will have to empathize with the people they lead. Communication and continuing education are and will continue to be part of the process of digital transformation in construction. Continuing education will be ongoing, more frequent, and effective in the lives of workers in the construction industry.

**LEAKY FEEDER**

Until recently, the most advanced tunnel communications solution was leaky feeder. It has long been the communication system of choice. This is not due to the productivity it offers, nor due to the quality or convenience of the system itself. Leaky Feeder had become the standard primarily due to its early availability and that it met the minimum requirements of a communication system.

The reality of the Leaky Feeder system is that it is essentially the low-tech equivalent of running a continuous antenna throughout an underground construction system.

One advantage of a leaky feeder cable is that it can be laid down as the tunnel is being excavated. Splitters can be used to send lengths of cable down different pathways. And because cable is flexible, there's no problem moving the network around sharp corners and turns. The cable can be fed straight down a hole if need be.

There are a couple of downsides to leaky feeder systems. A primary and ongoing issue with the Leaky Feeder system is the ability of the signals to propagate with maximum efficiency, and the antenna needs to be mounted low, exposing the Leaky Feeder system to damage from mobile equipment. If something severs the cable, communications stop beyond the break.

Another problem is that multiple leaky feeder cables can sometimes cause interference within the system. Leaky feeder radio frequencies tend to be on the high end of the spectrum-- these high frequencies don't penetrate rock very well.

Wi-Fi networks are an alternative to leaky feeders. A Wi-Fi network doesn't require a cable like a leaky feeder system. A fiber optic cable can be a backbone or be a wireless mesh network with no need for cable at all. Let's take a closer look at how wireless networks may make tunneling safer.

**Wi-Fi TECHNOLOGY**

The most useful and up to date digital technology for tunnel projects are Wi-Fi based systems (Figure 1). These systems provide a unified communications and data networks that allow the following:

- Two-way voice and texting via Voice over Internet Protocol (VoIP) portable handsets (Phones).
- Provide real-time location of people and equipment in the tunnel.
- Data communication (for TBM, roadheaders and other equipment).
- Interface to existing radio systems to allow the systems to “talk to each other”.
- Remote access to systems
- Data acquisition
- Video surveillance system in the tunnel.

There are Wi-Fi networks designed to lead tunneling communications and digital network infrastructure into the future. These network infrastructures have been specifically developed for the tunneling industry to operate within the harsh environments encountered in all underground operations.
The underground network is the heart of a scalable, high-speed data and communications system. It can cope with time-sensitive, high-bandwidth applications, enabling functionality such as Voice over Internet Protocol (VoIP), video streaming, remote PLC programming, mobile data acquisition, real-time vehicle diagnostics and asset / personnel tracking. Wi-Fi systems deliver improved capabilities for current and future tunnel requirements through higher reliability and support for open standards.

The Wi-Fi network can converge all data onto a single network. There is no need to run separate optic fiber or copper data lines to get data from critical equipment, such as TBMs (Tunnel Boring Machines). There are future proof systems for voice communication and tracking, with up to 1 Gb bandwidth, more than enough capacity to handle all the data requirements of a typical tunneling operation.

A typical high bandwidth network topology is shown in Figure 2. This shows the logical connections between the various IT and OT applications, which can be utilized in an underground project. The core infrastructure components that comprise the communications platform are:

- Wireless network switches
- Wireless access points
- Composite cable
- Antennas
- Communications appliance

![Figure 1. Wi-Fi based system for twin tunnel](image1)

![Figure 2. Typical IMPACT Installation for SCADA, tracking, voice and video applications](image2)
Wi-Fi Platform, Key Components

*Wireless network switch*

The ‘workhorse’ of the wireless infrastructure platform is the wireless network switch (Figure 3). The switch design may vary. In general terms, it consists of four managed, fiber optic Ethernet switches, two 802.11 b/g (IEEE) wireless access points, and four Power over Ethernet (PoE) ports providing scalable wired and wireless network access.

It is a multiservice device to support automation systems and/or facilitate functionality such as VoIP, IP video streaming, remote PLC programming, mobile data acquisition, real-time vehicle diagnostics, and asset/personnel tracking.

There are switches with a rugged enclosure designed for the underground environment and hence do not require a separate environmental enclosure. They are installed directly onto the back or side walls of the underground drives.

*Fiber optic cable*

Whether we use a single mode fiber optic cable (cable has a small diametral core that allows only one mode of light to propagate) or multimode fiber optic cable (cable has a large diametral core that allows multiple modes of light to propagate), we can use it as a backbone for a Wi-Fi installation throughout the tunnel.

Most underground environments have enterprise networks with a star topology, which requires power at every network node. The challenge of limited power availability can be overcome by using composite fiber cable, which carries both power and the single mode optical fiber cores.

This seemingly simple concept is a key to simplifying installation and maintenance of the network to semi-skilled labor. By incorporating both optic fibers and copper power cores so that power is transferred along the network by the single cable, the number of power insertion points required is greatly reduced.

*Communications appliance*

The ability to remotely manage and monitor the network is critical to any network deployment. A server based hardware and software platform that constantly monitors the network and provides tools for managing the individual network components is required. There are many software applications that display tracking, voice communications, machine health data, system alerts etc.
**Antennas**

There are several tunnel specific antenna designs. The most appropriate design depends on the tunnel alignment. Antennas can be easily installed or relocated as required. Depending upon the final antenna propagation testing, you can also deploy other models of antennas to ensure saturation coverage for the tunnel. Figure 5 is an example of network prediction model inside a long and curved tunnel using propagation software.

**Wi-Fi Technology Applications**

**Voice Communications**

Voice over Internet Protocol (VoIP) phones (Figure 6) can connect directly to the tunnel and surface network using wired Ethernet or Wireless coverage. Every wireless network switch and wireless access point (WAP) can be used to carry voice calls and messaging. The network solution is installed as a voice ready system and could be integrated to the site IP Private Branch Exchange (PBX) phone system if it has Session Initiation Protocol technology (SIP) trunk capability. This would create a seamless network for voice communications when operating on site. Restrictions could be enabled based on each user and call groups established as required. This would provide the following call types:

- Call direct to any user
- Push to talk to any user
- Programmed channels
- Emergency broadcast channels
- Calls to outside line

VoIP phones that also have a push to talk (PTT) button can be easily found on the market and are able to communicate to many customisable PTT channels as well as a broadcast channel.

**Tracking People and Equipment**

Tracking Systems report the location and track the movement of people and equipment as they travel through the tunnel in real-time.
Figure 7. Plan View of Personnel & Equipment in Tunnel

The tagging and tracking systems use strategically placed Wi-Fi Access Points as readers for RFID Tags. These are normally carried by workers or attached to vehicles and other equipment, so their real-time position is monitored underground and on the surface (Figure 7).

Many of the RFID Tags have a “MAN DOWN” button. If the person holding the tag presses the button, an emergency alarm will be sent to the control room, the location will be known and the spot on the tracking software will be turned RED.

There are many systems that analyze and interpret location data to deliver measurement reporting and comparison against theoretical values, to allow the real-time management of various critical production processes. In tunneling, if any person or mobile equipment fall outside pre-set parameters for time and location, an alert can be raised to investigate the issue and manage corrective action, before any delay to the process becomes significant. The most common rules to trigger alarms are:

- Breach Site Rule: This alarm triggers if a tag enters or exits a specified restricted zone.
- Distribution Site Rule: This alarm triggers when too many are in a defined zone (such as the TBM).
- Out of Range Site Rule: This alarm triggers when a person has been out of zone and not come back.
- Motion Stop Site Rule: This alarm triggers when a person has been stationary for a long duration.

The functionality could be extended beyond simple location awareness, so in addition to the safety benefits, an investment in a Wi-Fi Tracking System can deliver tangible cost benefits.

**Electronic access control**

There is a wide variety of electronic access control systems, though most of them use a rugged computer kiosk. These Kiosks (Figure 8) send the read information to the server and control room, either wirelessly or wired into the network.

![Kiosk](image)

**Emergency Call Points and Sirens**

To facilitate a dedicated fixed emergency alarm system, IP Help Points (Figure 9) are the best solution. These help points feature a single emergency call button. This button can call the control room or even broadcast the message to all connected devices. It provides direct communication between the control room and the IP Help Point.
Help points may contain two relay outputs. These relays can be controlled by the master control. Emergency sirens can also be installed to the call point, as shown in Figure 9. These sirens have configurable emergency tones and volume levels. From the control room, groups of emergency alarms can be trigged as deemed necessary and appropriate to the situation. For example, the group could be set up to trigger alarms for the complete evacuation of the tunnel in case of an emergency.

**WI-FI NETWORKS DESIGNS**

There are serval configurations to install a Wi-Fi platform throughout. The selection of one or the other depends on the tunnel communication requirements. Fiber based solutions are the most powerful and most reliable. There are many occasions where a wireless mesh network could be very cost effective.

**Fiber based Wi-Fi platform**

These advanced IP data networks deliver the following operational technology benefits during the initial construction, excavation and tunneling operation:

- Multilayer IP network to accommodate all tunnel operational technology
- Rapid deployment networks from the initial surface construction period
- A stable and reliable underground electronic tagging solution that is industry proven
- Voice & Text communications and Emergency Evacuations Solutions
- Video Surveillance solution that can operate on the surface to the tunnel face
- Live Data connectivity

A server stack will be installed at each key location and provide the network depending on their specific requirements and project stages. The primary components to be installed in an IT communications rack are the server, IP VHE (Voice Head End) server, RoIP (UHF/VHF - Radio over IP) Radio Bridge, IP surveillance server and network switch.
This operational technology IP ecosystem will provide the platform for all IP technologies to connect and ultimately provide the valuable visual content onto a software. All the individual site control systems have the ability to interconnect with all other sites creating one view for the entire project operation. This information can also be displayed back at any head office location, training facility or on secure mobile device.

QoS (Quality of Service) is provided on the Virtual Local Area Network (VLAN) and is administrated in accordance with the project IT policies. This allows management of all the applications and prioritizing for the data, voice, video and other traffic on the network via the one gigabit copper and fibre composite cable.

A Fiber based network can be designed to guarantee communication under any circumstance. If so, there is no need to have a different communication system in case of an emergency. By creating a loop with the fiber optic cable, redundancy is built within the system. This can be achieved in a solo tube tunnel, by running two fiber optic cables, or in a twin tunnel, by connecting both fiber cables. Next is an explanation of how the system behaves under any possible scenario in case of accident/failure for an installation in twin tunnels.

**Normal operation**
The Wi-Fi system could have the following services:

- Voice communication (VoIP phones + radios push to talk)
- Electronic Access Control
- Tracking (Personnel and vehicle location)
- TBM and Belt conveyor data
- Internet Access from all devices with Wi-Fi
- Video Cameras
- Wi-Fi platform providing all the benefits and services explained in this document

**Composite Cable gets cut**

![Figure 13. Twin tunnels fiber optic gets cut](image)

If the cable gets cut, the system will keep working as usual. The system will keep providing all the services, while the area where the cable has been cut will be identified immediately, and subsequently fixed.

The Wi-Fi system will keep offering all above-mentioned services.

**Server gets damage/destroyed**

In the unlikely event that the server gets damaged and stops working, the following services will remain in operation:

![Figure 14. Twin tunnels – Server goes down](image)

- Communication via PTT (VoIP phones and radios)
- TBM and Belt conveyor data
- Video cameras (CCTV throughout the tunnel will keep working)
- Wi-Fi platform providing all the benefits and services explained in this document

Another server could be installed as back up. In the unlikely case that the server gets damaged the backup one will be ready to work immediately and the voice communication via Wi-Fi will always be guaranteed. Desk phones could be installed throughout the tunnel to guarantee the Wi-Fi communication under any circumstance.

**In case both cables get cut**

Though extremely unlikely, in the event that the cable gets cut in both tunnels at the same time, the system will still be offering the following services:
Figure 15. Twin tunnels both cables get cut

ZONE A
- Communication via PTT (Push-To-Talk, VoIP phones and radios)

ZONE B
- Voice communication (VoIP phones + radios push to talk)
- Electronic Access Control
- Tracking (Personnel and vehicle location)
- TBM and Belt conveyor data
- Internet Access from all devices with Wi-Fi
- Video Cameras
- Wi-Fi platform providing all the benefits and services explained in this document

With any other communication system (leaky feeder, fibre optic, fix mine phones) if the cable gets cut, there is no possibility to communicate with Zone B, whilst by creating redundancy with a Wi-Fi system, Push-to-talk communication will always be guaranteed.

**Wireless Mesh Networks**

There are tunnel projects where full coverage along the tunnel is not needed, and people mainly work on the Tunnel Boring Machine (TBM) or in any particular area, including the surface. Under these circumstances, communication is mostly required at both job locations, and in case a temporary job needs to be done somewhere in the middle of the tunnel, wireless repeater nodes (WRN) could be used to provide temporary Wi-Fi coverage.

WRNs simplify the connectivity in the most dynamic areas underground, providing wireless connection to the working faces. A core requirement to leverage IoT is to have a reliable Wi-Fi connection. There are many WRN that meet this requirement and extend the Wi-Fi platform from the fixed infrastructure to the active working faces. These repeaters also include an 802.11n (IEEE) interface to allow easy access to the many Wi-Fi devices.

Figure 16. WRN

This solution allows different tunnel configurations, being the system so flexible, it is down to the project to choose the configuration that best meets the project requirements. Below are a few scenarios studied, using an 802.11n 2.4GHZ WRNs:
The figure 17 shows the best configuration to locate up to 7 Wireless Repeaters Nodes (WRN) in the tunnel. Under this design, up to 10 VoIP phones could be used at the same time to communicate from point A to point B with no problem at all. In other words, being all 10 VoIP phones making calls at the same time.

The figure 18 shows the worst configuration to have 7 WRN in a tunnel. Under this design, only 4 VoIP phones could be used at the same time to communicate from point A to point B.

This operational technology IP ecosystem will provide the platform for all IP technologies to connect. All of the individual site control systems have the ability to interconnect with other sites creating one view for the entire project operation. This information can also be displayed back at any customer head office location, training facility, or on secure mobile devices.

**Combination of Wireless Mesh Networks with a Fiber based Wi-Fi platform**

There are many circumstances, where only voice communication is needed. Yet, if there is a fiber optic cable in the tunnel, a very cost-effective solution is combining repeaters nodes with switches in the tunnel.

By using WAPs to create a mesh from switch to switch, there is no need for breaking the fiber frequently throughout the tunnel.

A server stack should be installed at a key location and provide the network systems for the site depending on their specific requirements and project stages. The primary components to be installed in an IT communications rack are the IP Voice Head End (VHE) server, IP surveillance server and network switch. These systems will provide important network services such as Wi-Fi for both voice and network access, establish the tag and tracking solution for the underground works and the like.

The design of the installation for the tunnel will be:
Figure 19. Combination of Wireless Mesh Networks with a Fiber based Wi-Fi platform.

This solution consists of installing a switch at the entrance of the tunnel, connected to the TBM’s fibre optic (or the tunnel fiber optic), and a switch installed at the TBM (or at the other end of the tunnel).

Approximately each 500 meters a WAP will be connected to the Low Voltage 240V network, extending the Wi-Fi signal throughout the tunnel. If power supply is not available in the tunnel, WRN can be used instead.

WAPs don’t need to be physically connected with one another, that’s why no cable is needed. It is a wireless solution that will provide Wi-Fi to the entire tunnel.

CONCLUSIONS

It has been a long-held assumption that because digital wireless systems are rooted in modern technology, those systems will be more expensive and therefore harder to justify than the leaky feeder systems. The reality is that because the Wi-Fi systems do use modern, proven technologies and materials, it is in fact price preferred when tracking and other expandable options are considered in the evaluation.

Wi-Fi system is a modern alternative, offering advantages that are not and cannot be available with the leaky feeder. These systems are generally 16% less costly than leaky feeder when tracking and voice are included in the system.

Traditional surface enterprise networks have a star topology which requires power at every network node. This is not a cost-effective solution underground. The challenge of limited power availability underground is overcome by using a composite cable, which acts as a power distribution system, as well as carrying the optic fiber data cores.

For a large-scale operation, the level of Wi-Fi coverage needs to be defined. Depending on the budget for its implementation there might be different options like:

- Fiber based Wi-Fi platform
- Wireless Mesh Networks
- Combination of Wireless Mesh Networks with a Fiber based Wi-Fi platform.

If there is a fiber optic cable in the tunnel, there is no more cost-effective communication system than Wi-Fi.

If only voice communication is required, mesh networks are positioned as the best solution for tunnels in length less than 3,000 meters.

REFERENCES

High Definition Maps for the future of Connected and Autonomous Vehicles

Autonomous Vehicles (AVs) are a hot topic right now. Most major automobile companies are projecting near-future timelines for commercial vehicles with advanced autonomy. There is also general agreement that, in order for AVs to become a reality, a High Definition Map must be created and maintained. There exist numerous challenges to achieving a usable High Definition Map including collection and processing of data, accuracy, ownership, update frequency, communication, insurance, and standards just to name a few.

In this presentation we will explore several of these challenges as they relate to the future computational requirements of AVs, and of the infrastructure required to support them.

- HD Map Definition
- How HD Maps are Created
  - Data Collection
  - Data Integration
- Why HD Maps are required
- How HD Maps will be used
- Opportunities and Challenges moving forward
IRF GLOBAL R2T Conference  
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KEYWORDS:  
Autonomous, impact protection, truck mounted attenuator, low speed operations, shadow vehicle.

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Royal Truck & Equipment Inc. is one of the largest suppliers of truck mounted attenuator trucks. Despite numerous safety innovations developed in the industry, drivers of truck mounted attenuator trucks still hold one of the most dangerous jobs in the country. Through partnership with Kratos Royal has developed the first autonomous truck mounted attenuator truck. Automating truck mounted attenuator trucks removes driver from the truck and eliminates their risk of injury or death. The Autonomous truck mounted attenuator truck is also significantly more efficient and precise than any human driver is capable of being. Safety and security precautions have also been implemented into the Autonomous truck mounted attenuator truck. Due to this being “icebreaker technology,” many states are eager to begin putting autonomous vehicles on the road. The Colorado Department of Transportation has already demonstrated the vehicle and is planning to implement it into standard operations soon. This technology has numerous applications, and Royal predicts that it will be seen serving a wider variety of roles as the technology spreads and is adopted.
The Challenges of Mobile/Stationary Workzones
Performing highway maintenance is extremely dangerous work because distracted, preoccupied drivers are navigating their cars or trucks at high speeds through stationary and mobile work zones. Mobile highway work zones are especially dangerous because maintenance vehicles are moving at a very slow speed in relation to traffic in an area that doesn’t seem like a work zone since sometimes it is only one or two vehicles painting lines in an otherwise normal highway. These slow-moving maintenance vehicles are generally protected from impact from the rear by a Truck Mounted Attenuator (TMA) also known as an Impact Protection Vehicle (IPV). An attenuator, also known as a crash cushion, on the protection vehicle saves the life of the person who makes the mistake of entering a work zone. The TMA truck also saves the lives of the crew in the work zone ahead by shielding them from the collision. However, this begs the question, what about the driver of the TMA truck? Until now, that IPV or TMA truck was driven by the lowest man on the highway worker totem pole. That person’s job was to drive the truck that protected everyone but themselves. Drivers of TMA trucks have lost their lives or sustained serious injuries that can affect them for the rest of their lives. This is a cost we should be unwilling to accept. Our role and mission as a truck manufacturer is to constantly increase the safety of highway workers by developing new technologies and safety features in our trucks.

TMA Best Practices
To have a safe Autonomous TMA (ATMA), the base TMA truck should be built to the following minimum standards that Royal promotes and that a few states have adopted. There is no federal regulation on the proper build of a TMA truck, it is up to the states. This three-part minimum standard ensures the TMA truck performs as expected when hit. The truck must have 20,000 lbs. total weight that ballasts and balances the truck to withstand the impact with minimal roll ahead distance. This weight was chosen for several reasons. According to the NCHRP Report 350, “risks to the occupants of the impacting vehicle generally increase as the mass of the support truck increases,” and “risks to the occupants of the support truck and to workers ahead of the truck generally increase as the mass of the support truck decreases” (Ross et al. 1993). By keeping the TMA truck at roughly the third quartile of weight for medium sized trucks, the TMA both protects motorists as well as workers. In addition, this ballast must be contained and not be released to cause more injury upon impact. A common violation of this principle occurs when an operator places “Jersey Barriers” on the bed to add weight. When hit, the concrete barriers break loose and undo the lifesaving work the attenuator just performed and goes directly against the criteria laid out for support vehicles in the NCHRP Report 350 (Ross et al 1993). The second necessary ingredient is the use of air brakes. Unlike traditional hydraulic brakes, air brakes are much more effective in holding the vehicle in place following a collision. Air brakes are specifically designed with numerous fail safes, such as air reservoirs, one-way check valves, dual air brake systems, an air compressor, pressure protection valve, and spring brakes, in place so that if there is a malfunction of the system they engage (August 2018). They will withstand impact and still hold the vehicle. The third ingredient is an attenuator to absorb the impact of the vehicle erroneously entering the workzone. There are many makes and models but only two are currently able to fulfill the new Manual for Assessing Safety Hardware (MASH) requirements for this class of traffic control devices. The TrafFix Scorpion II, an attenuator employed on most Royal TMAs recently met the criteria outlined in the American Association of
State Highway and Transportation Officials’ (AASHTO) Manual for Assessing Safety Hardware (MASH). In the 3-53 test of the Scorpion II, the TMA was only moved forward 20.3 ft (Ritter 2017).

**ATMA Added Benefits**

There are three major benefits to the ATMA system. The first and most significant benefit is the removal of the driver from harm’s way, including potential death or lifelong injury. According to “Analysis of Expected Crash Reduction Benefits and Costs of Truck-Mounted Attenuator Use in Work Zones,” nearly 35 percent of collisions with TMA trucks in New York resulted in injury (Ullman 2014). Employing the ATMA truck would reduce the casualty rate of collisions with TMA trucks. Second, in an ATMA truck the human instinct for self-preservation is removed with the driver. Automated vehicles will not flee oncoming errant vehicles as human drivers may do, leaving the lead vehicle and its crew unprotected. Third, by using an ATMA system human error is also removed with the driver. Automated vehicles maintain the gap distance with near perfect accuracy while human drivers will often come too close to their protected vehicle thereby endangering them with a secondary impact.

**How the System Works**

Mobile highway maintenance operations travel at a slow speed of 5 – 15 miles per hour and as such they provide a perfect use for this technology. The ATMA system is a leader / follower system where an unmanned TMA truck follows exactly where the lead vehicle goes. The system uses GPS and Real Time Kinematics (RTK) “e-crumbs” -Vehicle to Vehicle (V2V) communications from the manned lead vehicle to the unmanned following vehicle about position, speed, and heading. These e-crumbs are dropped at a rate of ten times per second allowing the following vehicle to travel with accuracy of +/- 4in. of the path of the lead vehicle.

Numerous safety precautions have been installed and implemented into the ATMA. Safety “e-Stop” buttons in both vehicles and on the exterior of the following vehicle will stop the ATMA when pressed. A Remote e-Stop, which is in the hands of the person assigned to monitoring the vehicle from the lead vehicle operates outside of the ATMA system and brings the ATMA truck to a controlled stop when activated. Any loss of connectivity will also stop the system. A Radar system detects an intrusion into the area between the trucks and stops the ATMA if something comes between them. An automatic sensor (longitudinal accelerometer) also detects impact from the rear and automatically applies the airbrakes, shuts off the engine and turns on the hazard lights.

The ATMA truck has been equipped with numerous Cybersecurity precautions to make the vehicle as secure as possible. Cybersecurity precautions include a combination of technologies, processes, and practices protecting the network, computers and data from attack, damage, or unauthorized access. The system computers and V2V communications are prevented from accessing any internal/external vehicle interfaces. Additionally, there are no Wi-Fi, Bluetooth, or cellular interfaces installed on the system, completely isolating it from external wireless networks to minimize cyber-attack vulnerabilities. Cyber security risks are further mitigated by limited physical access, vulnerability management, and system hardening techniques.
This autonomous system is “bolt on” technology; it can be added to existing vehicles and once added, it can be driven in either autonomous or manual mode.

**Current Deployments in United States and Worldwide**

Colorado Department of Transportation (CDOT) has been the first government agency in the US to adopt this technology. This past fall CDOT tested, launched, and deployed the ATMA under its RoadX program (Dougherty & Ford 2017). Following this highly successful preliminary run, CDOT has discussed with Royal plans to put one as a protection vehicle behind every kind of mobile operation within 5 years. This includes their current use behind line painting trucks and future uses behind sweeper trucks, cone setting trucks, behind roadside mowing operations in dangerous areas and crack sealing operations to name a few.

Colorado has also, through Federal Highway Administration’s (FHWA) Transportation Pooled Fund (TPF) Program initiated a solicitation on Autonomous Maintenance Technology (AMT) to support and promote collaborative research efforts in the field of autonomous technologies in work zone applications to improve safety, efficiency and quality of work efforts. There are currently 10 partner states whose commitments to this endeavor to date add up to $650,000 (Reeves 2017).

The ATMA truck is also beginning to see deployment abroad. Royal has partnered with Colas, an international highway contractor in England, and Highways England, the maintenance arm of the UK Department of Transportation, to get the ATMA truck implemented into highway operations in the United Kingdom (Ellrichpuram 2017). Royal is also currently in talks to have the vehicle deployed in other countries as well.

**ATMA as Icebreaker Technology - Advance Highway Safety and Early Adoption of Innovations**

The intense interest in this system can be attributed to the fact that this technology will save lives and because this is “ice breaker” technology that is easily adopted without any infrastructure improvements. It operates at a slow speed (5-15mph) in a controlled environment, the highway workzone. States can take a small step to utilize new innovations and save lives at the same time. Ice breaker technology appeals to everyone and opens the door to potential acceptance of other uses. It can help the public see the possible uses of autonomous technology. This could lead to greater acceptance to autonomous vehicles and to a lowering of highway fatalities.

**Future Additional Uses for this Technology Attainable Within 2 years**

The potential uses for ATMA technology in mobile operations are countless. In the next 2 years we predict that there will be many other mobile operations besides line painting such as road sweeping, rolling lane closures, roadside mowing that will use ATMAS. ATMAS could also begin to be used for “stop and go” operations like highway storm drain cleaning, highway light bulb changing, bridge “snoopers” assessing bridge condition, and debris removal operations. ATMAS with offset capabilities could run behind another vehicle while being offset to stagger the protection on teamed vehicles.
Conclusion

Despite numerous developments made to TMA trucks to save lives and reduce injury in automotive collisions, TMA drivers still have one of the most hazardous and life-threatening jobs in the country. Through the implementation of the ATMA across the country, Royal hopes to remove the driver out of the equation in these dangerous collisions. ATMA technology also makes shadow trucks significantly more effective at preforming their job. ATMA technology makes these shadow trucks more accurate than human drivers, are implemented with numerous safety precautions, and are secure from cyberattacks. Despite how new this technology is, ATMA’s are already being deployed in the United States due to how accessible this technology already is. Royal predicts that the use of ATMAs will only increase as time goes on. Autonomous vehicles are the future of roadwork safety, and the ATMA is currently paving the way in the industry.

Acknowledgements

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Multifunctional Infrastructure Corridors for Operating Electric Vehicles with Different Degrees of Automation

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**KEYWORDS:**
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**ABSTRACT:**
Intelligent, multifunctional infrastructure corridors, where vehicles are driven on different lanes depending on the system features or use platooning, operate automatically on separate highway sections or can orderly switch between sections with different operating forms, are an important prerequisite for introducing highly automated driving scenarios for road traffic. Suitable sensors collect traffic and environmental data along the infrastructure corridors at fixed and mobile points to locate the highly automated vehicles along the corridors. The eco-friendly electrical energy needed for the electric vehicles is generated, fed and supplied locally to energy stores along the corridors for charging purposes (centers, parking lots, lanes) from wind turbines located next to the freeways or various cross section elements with solar panels. A research project is conducting experiments on the layout and dimensions of these corridors and an energy balance sheet is being prepared for a highway section based on the vehicle mix (fuel-operated/electrically-powered).
1 INTRODUCTION

Extensive development work is taking place in the vehicle industry around the world in order to be able to manufacture vehicles with electric and hybrid drive systems so that they are ready for the market and can be produced on a large scale. There are, however, only a limited number of mass-produced electrically-powered vehicles on the market at the moment.

German automobile manufacturers plan to bring electric vehicles suitable for everyday use on to the market in 2019. In order to be able to use electric vehicles efficiently and in large numbers at the same time, significant further developments in the existing infrastructure will be necessary in the medium term – i.e. the existing and new traffic infrastructure will have to increasingly support electromobility, taking into account specific road and ecological requirements.

As traffic infrastructure in Germany is largely financed by the federal and state governments and there is already a significant logjam in maintenance work on existing networks, there has only been limited research work in this specialist field.

In the light of the long planning and approval periods for traffic infrastructure in Germany (approx. 15 – 20 years), it is necessary to ensure that the future upgrading work standards keep pace with the development of electric vehicles. The multi-functional infrastructure corridors required for this necessitate further developments so that the roads become intelligent roadways, which not only provide a means of pure mobility, but also energy generation, energy transport and energy transmission.

2 THE AIM

The new kinds of requirements for intelligent and ecologically sustainable road-oriented transport infrastructure have been identified as part of an interdisciplinary feasibility study [1] at Zwickau University of Applied Sciences. Scientists have examined how to gradually introduce the new technologies within a selected pilot scheme using different scenarios. Particular attention has been paid specifically to combining the requirements for roads, energy and ecological issues.

- Specific road requirements
  - Optimizing the route’s longitudinal profile (a good ratio between uphill and downhill sections) to reduce the resistance caused by inclines (alternating longitudinal profile),
  - Changing the design of the cross section by linking charging lanes at various points,
  - Reducing the rolling resistance and the rolling noise by providing suitable road surface coatings.

- Ecological and energy requirements
  - Using the route corridor for traffic infrastructure, energy generation equipment with supply networks (optimization of the corridor dimensions and pooling transmission routes),
  - Generating energy in the corridors by using the adjacent free space and selected cross section elements in the road in order to install solar power elements and wind turbines
  - Energy transmission in the corridors through using parallel supply networks to the necessary energy storage places,
  - Energy infeed at individual points or along sections through charging points and through parallel charging lanes along certain sections of the route.

Figure 1 illustrates the basic functional design of a multi-functional corridor for a highway. In addition to the 6-lane road, there are charging lanes, adjacent free space for energy generation equipment
(fields with solar power units, wind turbines), supply routes for transporting energy and rail tracks. Cross section elements on the highway (e.g. lanes, median strip, embankments, noise protection walls etc.) can also be used for generating energy, depending on their technical feasibility.

Figure 1. Macroscopic infrastructure corridor with traffic installations, energy generation units and supply networks

3 THE CURRENT STATE OF SCIENTIFIC KNOWLEDGE AND TECHNOLOGY

A study of the literature available [2] has shown that the standard rules for highways and rural roads in the major industrial countries do not take into account any special requirements for the alignment and design of the cross section as a result of electromobility at the moment. There has been no all-round interdisciplinary study into possibly linking electric vehicles with the infrastructure – i.e. the transport infrastructure currently only operates as a roadway for moving vehicles. Any basic studies into the use of cross section elements and special areas in the corridors for energy generation purposes, transporting energy and storing and transferring energy are not available either. There have only been some individual studies, e.g. on using noise protection walls with integrated solar-powered elements to generate energy [3].

A prototype for a hexagonal solar cell element [4] for integration in the road surface as a modular road surface element has been developed in the USA and selected pilot programs have already been successfully tested, mainly at depots and parking lots.

There have been no studies on the possible energy balance for the new kind of route corridors depending on the power generation possibilities and consumption as a result of the number of electric vehicles on the road either.

The fundamental attitude to this area of research in the transport sector in Germany is primarily dominated by the current situation as it is – i.e. it would only be possible to plan this in economic terms for the highway network when constructing new routes in sparsely populated regions in the medium term. However, there are new opportunities to develop ecological transport infrastructure for developing countries, which are currently dealing with basic planning and the gradual construction of a highway network.

Because of the need to provide an all-round approach to electromobility, further studies for planning, designing, building and operating route corridors for generating, forwarding, storing and transmitting energy are urgently needed.
4 METHODOLOGY

Figure 2 illustrates the basic sequence for searching for a corridor.

The corridor route searching process can only take place if there is a set study area, normally based on a digital topographic map. The boundaries of the study area are the result of a previously performed regional planning procedure with coordination from all those involved in the planning work and the people affected. The following principle applies: “The study area should be as large as necessary.”

As a result of the subsequent environmental study (environmental compatibility study), an area resistance map is created, which establishes and illustrates in graphical form the resistance or the potential for the planned transport infrastructure to intrude, taking into account animals, humans, soil, water, air, climate, landscape, culture and material goods, which all require protection. “Red” (shaded) areas have a very high level of resistance and “gray” areas low resistance to a possible corridor track. The area resistance map provides the areas of least conflict when searching for a corridor (Figure 3).
Figure 3. The corridor provoking the least conflict using a computer-supported search for a route

5 MODEL PRESENTATION
5.1 Tangent method

The route search in the areas of little conflict takes place using the tangent method (Figure 4) with support from computers. The interpolation points $P_j, P_j \in \mathbb{R}^3$ of the tangent polynomial are set in the three-dimensional area, taking into account the restrictions. The associated corridor is automatically generated resulting from the set width. The automatic smoothing process then takes place with the help of Bezier curves. The designed corridor consists of a sequence of straights and curves. By shifting the manipulation points $Q_j, Q_j \in \mathbb{R}^3$, it is possible to change the geometric design of the corridor. An iteration process is used, taking into account the constraint areas. The polygon points $P_j$ must be varied to develop other corridor options and the iteration process used once again.
Using a system of coordinates, it is possible to manipulate the route in real time.

5.2 Mathematical approach

The following is valid as a general interpolation approach for the complete bend:

\[ X(t) = \sum_{j=0}^{n} A_j B_j(t) \text{ with } X(t_i) = P_i, \quad i = 0(1)n \]  

(1)

- \( A_j \): coefficients \( \in \mathbb{R}^3 \)
- \( B_j(t) \): basic function \( \in \mathbb{R} \)
- \( P_i \): Interpolation point \( s \)
- \( t_i \): Interpolation parameter

Integral Bezier splines are used because of the numerical instability and tendency for polynomials of a higher degree to oscillate and they make it possible to map bends and straights. The Bezier splines are calculated with the help of Bernstein polynomials. They are defined as follows:

\[ 1 = [(1-t) + t]^n = \sum_{k=0}^{n} \binom{n}{k} (1-t)^{n-k} t^k \]  

(2)

Summands: \( B_k^n(t) = \binom{n}{k} (1-t)^{n-k} t^k \)  

(3)

The recurrence equation comes from the definition:

\[ B_k^n(t) = (1-t)B_k^{n-1}(t) + t \times B_{k-1}^{n-1}(t) \]  

(4)

Proof:

The following is valid: \( \binom{n}{k} = \binom{n-1}{k-1} + \binom{n-1}{k} \)

(Pascal triangle, proof with complete induction)

\[ B_k^n(t) = \binom{n}{k} (1-t)^{n-k} t^k = \left[ \binom{n-1}{k-1} + \binom{n-1}{k} \right] \times (1-t)^{n-k} t^k \]

\[ = (1-t) \binom{n-1}{k} (1-t)^{(n-1)-k} t^k + t \times \binom{n-1}{k-1} (1-t)^{(n-1)-(k-1)} \times t^{k-1} \]

\[ = (1-t) B_k^{n-1}(t) + t \times B_{k-1}^{n-1}(t) \]

q.e.d.

The derivation of the Bernstein polynomials is defined as follows:
\[
0 \leq t \leq 1
\]

The Bezier splines are described as follows:

\[
X(t) = \sum_{i=0}^{n} b_i \cdot B_i^n(t), \quad b_i \in R^3
\]

The Bezier points are represented by \( b_i \). \( B_i^n(t) \) are the Bernstein polynomials already mentioned. It must be possible to differentiate the Bezier spline twice constantly at the interpolation points for the corridor design.

Studies (7) have shown that the technical design features required for the bend areas are best met by 5th degree Bezier splines (Figure 5).

![Figure 5. Providing transition curves to the tangent polygon with Bézier curves](image)

6 PILOT SCHEME
6.1 Marginal conditions

A stretch of highway measuring 10 km served as the pilot scheme. Its horizontal and vertical projections and cross section were designed in line with the Guidelines for Designing Highways (RAA). The maximum and minimum speeds for the design elements on the horizontal and vertical projections were followed. The vertical projection had an alternating longitudinal profile. The RQ 36 standard cross section (6 lanes and 2 breakdown lanes) provides a crown width of 36 m (Figure 6). In addition to the breakdown lane,
a separate charging lane, which is 3.50 m wide, was arranged along certain sections to enable vehicles to recharge their batteries during their journey if their energy supplies are low. Stationary charging pillars and lengthwise charging lanes complete the overall charging infrastructure. 40 m was set aside as adjacent free space on both sides. This means the corridor width is approx. 140 m.

The traffic volumes was estimated to be 4,500 vehicles per hour in each direction. It was assumed that the vehicles would travel at a constant speed of 70/100/130 km/h.

Several options were investigated using the corridor finder (8) and a preferred corridor was selected (Figure 7).

![Figure 6 Areas of energy generation using cross-section elements and in the adjacent free space](image)

![Figure 7 Preferred corridor](image)

Energy studies were performed too as part of routing the corridor – i.e. an associated energy balance was calculated (needs/yields), depending on the influencing factors.
The following 3 cases were observed when estimating the energy needs:

- Case 1: 0.01% electric vehicles (current)
- Case 2: 1.90% electric vehicles (2020)
- Case 3: 100% electric vehicles (extreme case)

A Mitsubishi i-Miev was used as the reference vehicle for the comparative tests (calculations, simulation, real journeys). The results obtained were compared to a study conducted by Vienna University of Technology (9). The following basic scenarios were used to determine the energy yields:

**Scenario 1:** Theoretical approach (maximum theoretical installable power)
- Large wind turbines
  - on the edge of the adjacent free space (one side) at a distance of 500 m
- Solar-power units
  - as the road surface: Solar roadway
  - above the road: Overhead solar units
  - in the median strip: Single module
  - on adjacent free space: Module fields
  - on noise protection equipment (wall/embankment): Module areas

**Scenario 2:** Real approach (maximum installable power that is technologically feasible at the moment)
- Large wind turbines (20% of the max. installable power)
- Solar-power units (50% of the adjacent free space)

Taking into account these terms of reference, the energy yield and needs were determined and contrasted for the pilot scheme in order to establish a simplified energy balance, depending on the conditions being examined.

6.2 Energy yield potential for solar power units

The studies conducted were assessed and analyzed. The following Figure 8 illustrates the maximum solar power output that can be installed for the highway, which runs in an east-west direction. The results clearly reveal that the “solar roadway” and “overhead solar units” scenarios provide the greatest yields, but the fundamental technical and technological principles do not yet exist for them.

![Figure 8](image-url)

**Figure 8.** Seasonal course of electrical energy yields with the maximum installable solar power output for a section of roadway running in an east-west direction
The approach already adopted of arranging fields of solar power units in the adjacent free space near the infrastructure corridor is technologically feasible and cost-effective too. The use of the median strip and any noise protection facilities that are required for energy generation purposes is, however, uneconomical and is therefore not to be recommended (1, 5).

6.3 Energy yield potential for wind turbines

A simulation process with a subsequent field experiment was used to check whether the currents of air caused by the movement of traffic can be used to generate energy with small wind turbines (along or above the roadway). The results (7) clearly revealed that the currents of air that are caused are very low and have no economic potential for exploitation, as the start-up speed of the units is 3 – 4 m/s.

For the purposes of energy generation in the corridor, a decision was therefore made to use 3 MW large wind turbines. The units were arranged at intervals of approx. 500 m on one side of the edge of the adjacent free space. It was possible to achieve annual power output amounting to approx. 10.8 GWh from the 3 MW units. This corresponds to about 3,600 full-load hours.

Figure 9 illustrates the results of the calculations for the energy yields in the corridor. In comparison with the solar power units, the energy yields are much more evenly spread over the year. If we compare the energy yields in the real scenario, the annual yields when using 3 MW wind turbines are approx. 170 GWh, while the solar power units in the adjacent free space only provide 34 GWh. As a result, wind turbines can provide the lion’s share of the annual energy yields.

![Figure 9. A comparison of the annual cycle of wind power yields with different output levels in terms of the total annual output (3 MW unit)](image)

6.4 Energy balance (energy requirements/energy yields)

An initial energy balance was drawn up, which is summarized in Figure 10, using the wind turbines and solar power units on the pilot route for the theoretical and real scenarios and the energy requirements calculated for the electric vehicles, depending on the driving resistance forces, the traffic density, the conditions of movement and the number of electric vehicles on the road.
Taking into account the outline conditions and simplifications that have been made, it is evident that if the maximum installable power (theoretical scenario) approach to energy requirements is adopted, it is possible to cover needs at all three speeds. However, energy generation using the “solar roadways” or “overhead solar units” is still not a feasible solution from a technical or technological point of view and therefore does not represent a cost-efficient option at the moment. However, the technology for energy generation with the help of large-scale wind turbines is available and does make economic sense.

When adopting the real scenario (i.e. 20% of the max. installable output from wind power and 50% of the max. installable output from solar power units in the adjacent free space), it is clearly evident that a positive or negative energy balance is created, depending on the speed and proportion of electric vehicles on the road.

Figure 11 illustrates the selected energy generation scenario in the infrastructure corridor. Electric vehicles are recharged using the charging lanes and charging points, which are linked to the public electricity grid via distributor transformers. Solar power units and wind turbines are located in the adjacent free space or what are currently the edges of the highway and they cover the energy needs of the electric vehicles in different ways. Energy storage units can be used to optimize the energy balance. Any excess energy from the energy generation units must be fed in and can be used in the network to compensate for supply peaks, if required.
7 THE RESULTS AND PROSPECTS

In order to introduce electromobility in normal transport operations in a comprehensive and all-round manner, it is essential to not only develop electric vehicles that are suitable for daily use, but also set up a new kind of supporting, intelligent traffic infrastructure.

In order to generate the necessary energy for electric vehicles ecologically, very close to the place of consumption and make it directly available, it will be necessary to plan and build multi-functional infrastructure corridors.

The software tool “Corridor Finder” has been developed to plan the corridors in a similar way to finding the correct route for a road; this enables the engineer to design the development and energy requirements for the corridors with all the equipment and dimensions interactively.

Energy generation using wind power and solar power units can take place in these corridors, although the energy balance will largely depend on the technological opportunities for ecological energy generation and the actual proportion of electric vehicles on the road.

Energy generation using large wind turbines and fields of solar panels in the adjacent free space is already feasible as a cost-effective method in a real scenario and can already be used to cover about 30% of energy requirements.

The direct integration of new kinds of solar power modules in the cross section element of transport infrastructure has not yet been technologically resolved, i.e. development work still needs to take place in this field.

The results of the studies carried out within the feasibility stages illustrate the high level of complexity, inter-disciplinary approach and the high technical, technological and cost-effective requirements for the overall subject matter of corridors – i.e. traffic and electrical engineers need to work closely together right from the planning stage.

It is absolutely essential that the responsible public building authorities press ahead with joint and more intensive fundamental research so that the new kinds of requirements can be used in the all-round
planning and design process for future traffic infrastructure in the medium term. It will also be necessary to gradually adapt the standard rules to the new requirements.

REFERENCES


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**KEYWORDS:**
Proximity, Safety, Bluetooth, Equipment, Roadway Construction

**ABSTRACT:**
Over the last decade, researchers have improved passive regulations for construction site safety. Despite these passive regulations, occupational fatalities frequently occur. To overcome the limitations of passive regulations, researchers have devoted a great deal of effort to developing active sensing and alert systems that triggers an alarm at a certain distance. Recent studies have shown that differences between the measured and prescribed alert distances are sensitive to types of equipment on which the system is installed. However, few studies have attempted to minimize the bias between measured and prescribed alert distances. To address this issue, this work introduces a parameter adjustment function that calibrates Bluetooth low-energy (BLE) proximity alert and sensing systems. Field trials have demonstrated that the calibration of BLE systems using the function improves system accuracy and stability under various types of equipment.
Parameter Adjustment Function for Bluetooth Low-Energy Sensors in Dynamic Construction Proximity Applications

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1 INTRODUCTION

Construction is a hazardous process that leads to injuries and fatalities. Over the last few decades, researchers, supported by the construction industry, have focused their concern on the safety of construction workers and improvements in the conditions that threaten their safety at road construction sites. In particular, safety conditions at road construction projects that lead to the second highest number of fatalities in the construction industry, have been promoted and enhanced by passive regulations, such as signage, traffic controls, flaggers, and other measures. Despite these efforts, however, the number of occupational fatalities and injuries has remained constant (Pegula 2013).

To reduce accidents caused by collisions, researchers have used several sensing technologies, such as radar, video camera, radio frequency identification (RFID), magnetic field detection, and Bluetooth low energy (BLE) to provide proximity alerts under hazardous conditions (Carbonari et al. 2011; Kim et al. 2016; Li et al. 2016; Marks and Teizer 2012; Park et al. 2015a, 2016; Ruff 2007). Although many of the developed systems based on these technologies provide proximity alerts under certain circumstances, some show inconsistency of alert distances for different types of equipment on which the system is installed. By conducting coverage experiences for various types of equipment with the BLE, the RFID, and the magnetic field-based proximity sensing and alert systems, Park et al. (2015a) found that alert ranges of proximity alert and sensing systems with same settings were inconsistent when they tested a wheel loader instead of a truck. Marks et al. (2012) encountered similar issues. Other studies find that in addition to the type of equipment, parameters, such as ambient temperature and relative humidity, impact the range of proximity sensing and alert systems (Marks and Teizer 2013; Park and Cho 2017). However, the inconsistency in the alert range has not received a great deal of attention in studies of the proximity-sensing domain, indicating an urgent need for investigation.

2 OBJECTIVE AND SCOPE

This study aims to develop a parameter adjustment function that overcomes inconsistencies in the detection range of the proximity sensing and alert systems for various types of equipment. The developed parameter adjustment function focuses on generating initial parameters based on raw data from equipment on which the system is installed. To assess the performance of the developed function, the authors have conducted field trials using various types of equipment. The scope of this paper focuses on reducing the inconsistency of the proximity sensing and alert systems used for various types of equipment. Even though many other factors may influence the accuracy and reliability of the proximity sensing and alert systems at construction sites, they are not the focus of this paper.

3 METHODOLOGY

To overcome the inconsistency of the detection range of proximity sensing and alert systems, we propose a parameter adjustment function in this study; for distance computation, this methodology discusses how each parameter (Equation 1) is adjusted for each piece of equipment. The function is developed and tested on a proximity sensing and alert system based on BLE, which is a new technology released in 2010. The advantages of BLE, such as low cost and minimum infrastructure requirements, have attracted attention in several research domains (Palumbo et al. 2015; Park
and Cho 2017; Zhuang et al. 2016). However, research regarding BLE technology in the construction domain is limited. Park et al. (2015b) were the first to introduce the BLE-based proximity sensing and alert system. We use a similar system in this study to test the capability of the parameter adjustment function to increase the consistency of the alert range under various types of equipment. A system architecture of the BLE system with the added function is shown in Figure 1, which includes system components, data flow, and function position.

![System architecture with an addition of the parameter adjustment function](image)

Figure 1. System architecture with an addition of the parameter adjustment function

As shown in Figure 1, the BLE system estimates distances between workers and equipment by using BLE signal communications between a personal protection unit (PPU) for workers that is a BLE signal receiver, and equipment operator units (EPU) for equipment that are BLE beacons attached to the equipment. As distances between PPU and EPU increase, the signal strength used to estimate the distance fades. The relationship between the received signal strength and the estimated distance is shown in Equation 1 (Chintalapudi et al. 2010; Li et al. 2005).

$$\text{Dist} = 10^{\left(\frac{\text{abs}(\text{RSSI}_1) + \text{RSSI}}{10n}\right)}$$

where $\text{Dist}$ is the estimated distance between a BLE beacon and a receiver device, $\text{RSSI}_1$ is a predetermined RSSI value measured at the distance of 1m, $\text{RSSI}$ is RSSI values measured in real time, $n$ is the path loss constant, and $\text{abs}$ gives the absolute value of RSSI.

The equation shows that the estimated distance is mainly based on the received RSSI value under constant test conditions. However, in reality, the received signal strength is not constant, even under the same test distance and condition. This inconsistency is mainly due to the dynamic nature of construction sites and various interfering conditions. To show inconsistency of the received RSSI values, we collect a set of data at a distance of 1 m using the BLE system without calibration. Figure 2(a) shows estimated distances based on received RSSI values and uncalibrated parameter factors. Figure 2(b) shows cumulative distribution of sorted RSSI values. Both figures present clear evidence that the measured distance base on the received RSSI value is not constant. In addition, neither the average nor the median of the estimated distance is accurate without calibration. To solve this issue, we develop the parameter adjustment function so that the proximity sensing and alert systems can adapt to unique conditions, such as various types of equipment.
Figure 2. Measured distances at 1 m without an adjustment of parameters

Figure 3 depicts the pseudocode for the developed parameter adjustment function. The first part of the algorithm is to calibrate RSSI for a specific type of equipment. We collect a certain number of RSSI data using the worker’s PPU device at a distance of 1 m to the EPU beacons. To reduce impacts of outliers, which are highlighted in Figure 2(b), we use only 80% of the collected data in the middle range to generate an average value that represents the calibrated RSSI. The second part of the algorithm is to calibrate path loss constant n. As the first part, we collect a certain number of data at a specific distance that is not equal to 1 m (e.g., 2 m, 3 m, or 4 m, etc). Outliers of the data that cause a large deviation of the results are also eliminated. Using the calibrated value of RSSI and the averaged value of the newly collected RSSI, we use back-calculate to get calibrated n in Equation 1. The application of this process to each BLE sensor creates more symmetric coverage of the alert distance and helps to adjust to different types of equipment.

4 FIELD EXPERIMENTS

To show the inconsistency of the BLE proximity sensing and alert system and the improvement of this issue using the proposed function under different types of equipment, we conducted a static field test by using a wheel loader and a dump truck. In the field test, a pedestrian worker approaches a stationary piece of equipment at a constant speed. The test scenario and test beds are shown in Figure 4.
Figure 4. Test scenario and test bed

Figure 5 and Figure 6 show system setup plans for the dump truck and the wheel loader. For each type of equipment, we test both the BLE system with and without the parameter adjustment function. During the test, a worker held the PPU at waist height and approached the equipment at a constant walking speed of 3 mph (4.8 kilometers per hour). When the distance between PPU and PPE was smaller than the preset distance of the system, alert was triggered. At this moment, the worker stopped walking, and we measured the distance between the closest part of the equipment and the worker. We measured eight equally spaced approach angles and repeated the test 20 times for each of the angles.

Figure 5. System setup with BLE sensors for the tested truck
5 RESULTS AND DISCUSSION

The results from the test without using the parameter adjustment function are shown in Figure 7. Sub-plots Figure 7 (a) and Figure 7 (b) show the results for the tested wheel loader and the truck separately. For each of the approaching directions, the graphs plot the average value and the confidence interval of one standard deviation. The comparison of the two graphs suggests that the alert ranges are not constant when the system is applied to different types of equipment. For example, the average alert distance of the wheel loader is significantly smaller than that of the truck. A possible reason for this inconsistency of alert distances between the two types of equipment is their different equipment configurations. Compared to that of the truck, the configuration of the wheel loader is more complex with more attached mechanical parts, which may block the line-of-sight signal communication or increase the impacts of multipath.

![Figure 6. System setup with BLE sensors for the tested wheel loader](image)

![Figure 7. Real alert distances of the BLE system without parameter adjustment](image)

Figure 7 shows the results of the test using the parameter adjustment function. Compared to the results of Figure 7, the alert distances are more constant for each type of equipment from different approaching directions. The
comparison of the two results presents no obvious changes of the alert zones. Thus, the proposed parameter adjustment function showed improvement in the consistency and reliability of alert distances for different types of equipment.

![Figure 8](image)

(a) Active proximity coverage of a wheel loader with parameter adjustment (10 m setting)  
(b) Active proximity coverage of a truck with parameter adjustment (10 m setting)

Figure 8. Real alert distances of the BLE system with parameter adjustment

The results from both conditions are summarized in Table 1. The absolute value of the difference between the average and the alert distance settings are used to reflect the accuracy of the system, which is shown as Function 2. Meanwhile, the standard deviations of the results are used to measure the consistency of data. Table 1 also shows that the proposed parameter adjustment function reduced the deviation from 5.54 m to 0.03 m for the truck, and from 0.88 m to 0.37 m for the wheel loader. In addition, the standard deviations for both types of equipment are reduced.

Deviation to the desired distance = abs(average distance - desired distance)  \[ (2) \]

Table 1. Summary of test results

<table>
<thead>
<tr>
<th>Equipment type</th>
<th>Parameter adjustment function</th>
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<th>Deviation from setting (m)</th>
<th>Standard Deviation (m)</th>
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<td>Wheel Loader</td>
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<tr>
<td></td>
<td>With</td>
<td>9.63</td>
<td>0.37</td>
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6 CONCLUSIONS

There has been a great deal of research to improve the safety conditions of roadway work zones using different sensing technologies. Even though the impact of different types of equipment on the alert distances has been mentioned in several studies, limited research has addressed this problem. In this study, a parameter adjustment function based on the BLE proximity alert and sensing system is developed. The test results from the field trials demonstrated that the parameter adjustment function improved the stability and accuracy of the BLE system under the circumstance of different types of equipment. This research is of value to researchers and practitioners that this particular method presents a method to reduce inconsistent performance of proximity warning systems.

REFERENCES


Assessment of Road Infrastructure advances for Mixed Vehicle Traffic flows: the INFRAMIX approach

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Connected and Automated Vehicles; Hybrid Road Infrastructure; Digital Infrastructure; Mixed Vehicle Traffic; Evaluation methodology

Over the last years, significant resources have been devoted to developing new automation technologies for vehicles, whereas investment for road infrastructure, in general, has steadily dwindled. INFRAMIX is preparing the road infrastructure to enable the coexistence of conventional and automated vehicles. Its main target is to design, upgrade, adapt and test both physical and digital elements of the road infrastructure, ensuring an uninterrupted, predictable, safe and efficient traffic. For that purpose, new advanced microscopic traffic flow models, advanced simulation techniques and innovative control strategies will be employed. In order to provide a clear impact, INFRAMIX developments will be evaluated on user appreciation, transport efficiency and safety performance through three high-value traffic scenarios: (1) Dynamic lane assignment, (2) Roadworks zones, and (3) Bottlenecks. Tests of the three scenarios will be performed in real-life conditions, at the project test sites, and also through extended simulations, especially in high penetration cases. Special attention will be paid to assess the users’ appreciation regarding the proposed information chain, the adequateness and understandability of visual and electronic signals, as well as the integrated control algorithms. Setting the respective research questions for the selected traffic scenarios, leading to new safety and performance criteria for mixed traffic, is an indispensable part of the evaluation methodology for such a complex and novel project. The evaluation methodology will be structured considering also the project objective of establishing an infrastructure classification scheme, which will set the basis for a timely deployment of an automation-appropriate infrastructure network.
Assessment of Road Infrastructure advances for Mixed Vehicle Traffic flows: the INFRAMIX approach

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1 INTRODUCTION

As more and more automated functionalities are incorporated in vehicles, the role of the road infrastructure in actively assisting them, has been vastly recognised. Although driveless vehicles have, in principle, been designed to operate independently using in-vehicle sensors, there are requirements that the road infrastructure should meet to enable their operation. (Zhang 2013), assigned infrastructure requirements to the different vehicle automation levels of NHTSA (National Highway Traffic Safety Administration 2018). Nowadays, road infrastructure is designed to accommodate for the circulation of conventional vehicles (human-driven vehicles). However, the role of the infrastructure is vital in managing the transition period when the penetration rate of Automated Vehicles (AVs) will gradually increase. In the near future, the infrastructure is called to enable and support the coexistence of conventional and AVs, accommodating for mixed traffic flows.

In this direction, the cooperative future, as it is envisioned by various researchers and automotive manufacturers, would not be possible without an active role of the infrastructure in the traffic flow management (Raposo et al. 2017), (Vantomme 2018). As a step further (Cheon 2003), made an infrastructure clustering in automation levels. In the first level, named “the infrastructure supported”, the road infrastructure supports vehicle decision-making while in the highest level, named “the infrastructure controlled” characterizes an infrastructure that has the full traffic control in all driving situations. Preliminary infrastructure requirements have been also set for a Coordinated Automated Road Transport (C-ART) future, as it is envisioned by (Raposo et al. 2017). Furthermore, in (Vantomme 2018), a cooperative road transportation future is presented where the infrastructure has an indispensable role. (Shladover 2017), points out the potential support that the road infrastructure could provide towards automating road transport. Apparently, developments in road infrastructure are necessary in order to realize the vision of automated transportation. The incremental upgrades of the road infrastructure will enable the coexistence of vehicles with different automation functionalities and will minimize the incidents of misuse of the automated functions related to their operational design domain, as defined at (SAE Standards 2014).

Despite the recognition of its vital role, the road infrastructure development is not proportional to the rapid deployment of the new automated vehicle’s technologies during the last decade. Consequently, the transportation research focuses mostly on the assessment of the novel automation functionalities (Barnard et al. 2015), (Innamaa & Kuisma 2018), rather than on the impact caused by road infrastructure upgrades.

Assessment of the impact that the road infrastructure development would impose in a potential mixed traffic flow is an endeavor, which implies multiple challenges. Firstly, the precise definition of such a complex system, that characterizes the road infrastructure which includes the required functionalities to accommodate conventional and AVs simultaneously, does not exist. Moreover, the lack of real data and existing statistics from interaction of automated vehicles with road infrastructure, due to the minimum AVs currently in the streets, is an additional issue. Another challenge is the fact that important parameters of a mixed vehicle traffic flow, such as the expected penetration rate of the AVs through the years, are based on assumptions. Additionally, a structured and concise evaluation methodology analogous to the ones for the automated functions (e.g. FESTA, (FESTA 2016)) is still under investigation.

This is exactly the purpose of this paper, the presentation of an evaluation methodology suitable for a “hybrid” infrastructure where different types of vehicles are driving. This methodology is currently under discussion within the EU project INFRAMIX1 and it is based on FESTA version 7 (FESTA 2016) originally developed for Field Operation Tests (FOTs), while addressing major transportation concerns which hassle the introduction of the AVs. The evaluation areas are the following: traffic safety, traffic efficiency, users’ appreciation and technical feasibility.

The “hybrid” road infrastructure in INFRAMIX is primarily designed to take care of three critical traffic scenarios (in terms of importance with regard to traffic efficiency and safety), without loss of generality (Lytrivis et al. 2018). These scenarios will be the basis for the impact assessment and evaluation:

1 https://www.inframix.eu/
• Dynamic lane assignment
• Roadworks zone / Construction site
• Bottlenecks

This work presents the hybrid infrastructure in the first section. The second section gives an overview of the traffic scenarios under investigation and the third one describes the evaluation methodology. The next section describes the future work, while the conclusions are drawn in the last part of this paper.

2 HYBRID ROAD INFRASTRUCTURE

While significant resources have been devoted to developing new automated vehicle’s technologies, investments and resources for road infrastructure have been, in general, decreasing in the last decade. Little work has been done on the way the infrastructure could support and handle the introduction of automated driving systems on the roads, while maintaining, if not enhancing, traffic flow efficiency and safety. On top of that, one has to consider three additional important factors: a) road infrastructure’s average lifecycle is well above (3-4 times higher) the average lifecycle of a vehicle, b) the high cost related to infrastructure’s construction and maintenance and c) the limited space available for building new roads (esp. in urban environments).

All the above converge to the fact that there is a need for deploying innovative technologies also at the infrastructure side. So, apart from the obvious adaptations, which are necessary at the level of the physical road infrastructure in order to cater for automated driving, corresponding digital advances, such as the creation of High Definition (HD) maps and new electronic signals, are also a must. This leads to the need for the provision of a “hybrid” road infrastructure (both physical and digital) which will be ready to cope efficiently with the new safety challenges emerging from the introduction of AVs and especially within the transition period.

(Lytrivis et al. 2018), highlights the need to design new and adapt existing physical or digital infrastructure elements (e.g. segregation, traffic signs, electronic horizon etc.) in order to allow the current infrastructure to address the introduction of automation in a flexible, fast and cost-effective way, while being understood by all traffic participants, automated or not. In order to achieve that, the “hybrid” infrastructure concept merges the physical and digital infrastructure into one system. Physical infrastructure consists of the roads, road signs, road markings, gantries, etc. that form part of the physical world where vehicles operate. At the same time, digital infrastructure is defined as the static and dynamic digital representation of the physical world with which the vehicle will have to interact (OECD/ITF 2015). For instance, high definition maps, dynamic traffic information and advanced advice related to optimum routing are some of the state-of-art technologies which are included in the digital infrastructure part.

A preliminary concept is conceived by the INFRAMIX project regarding the “hybrid” road infrastructure with time horizon 2020. Figure 1, depicts this concept for a traffic scenario when a lane is assigned to AVs.

![INFRAMIX high-level architecture](https://www.inframix.eu/)

Figure 1. INFRAMIX high-level architecture (source: H2020 INFRAMIX project 2018).
In order to provide an overview of the challenges on the implementation and assessment of such a complex concept, a closer analysis of its consisted elements is necessary. The following paragraphs give an overview of the current status of each component (depicted in Figure 1) and the required upgrades in order to address the challenges assuming mixed traffic flows within INFRAMIX project.

**Road infrastructure dynamic signage**

As the road infrastructure nowadays is built to accommodate conventional vehicles (human driven vehicles), the visual signs provide mainly static information (e.g. speed limits). Modern highways include also dynamic signage (e.g. traffic jam information, weather conditions warnings etc.) through Variable Message Signs (VMS). Nevertheless, this is only to be recognized by human drivers. For mixed traffic flow new visual and electronic signals that communicate information, issue warnings or provide guidance to all highway users (conventional and automated vehicles) need to be implemented.

In the “hybrid” infrastructure concept, the use of the current road infrastructure communication elements is investigated to facilitate the infrastructure-to-vehicle (I2V) communication with the vehicles which are not connected with the traffic management center (e.g. conventional vehicles). Moreover, novel signaling content related to innovative traffic management, like the lane assignment to AVs (see Figure 1), needs to be investigated. This is a challenging part especially for the evaluation, as the human drivers’ appreciation of new content of signaling plays an important role in the acceptance of novel traffic management functionalities. Another challenge, related to the development of the physical infrastructure, is the automatic and the real-time communication between the road infrastructure elements and the traffic management center (TMC). Currently, even in modern highways, the changes in the dynamic signaling is made manually by the road operator located at the TMC. This causes a delay which might be a limiting factor for dynamic traffic control.

Related to the in-vehicle signaling and guidance of the AVs (or to the vehicles which just have on-board equipment that permits V2I and I2V communication), different alternatives will be investigated such as nomadic and cooperative systems. To enable such systems road infrastructure should not only be equipped with Road Side Units (RSUs) (e.g. for the ITS-G5 network) but also should handle the challenges of sending at the same time a specific message to all users through different networks (e.g. LTE-V and ITS G5 as shown in Figure 1).

Considering a wireless bi-directional communication with the AVs, ITS specific wireless messages extensions are required. Therefore, the enhancement of existing messages like MAP, CAM, DENM and other C-ITS messages expected to be proposed (ETSI TS 103 301 2016), (ETSI EN 302 637-3 2014), (ETSI EN 302 637-2 2014). This is another challenge for evaluation similar to the one for the novel visual signs on the physical elements. In this situation, the assessment in matters of both users’ appreciation and technical feasibility (in the sense of implications to the AV operation) is necessary regarding the new wireless messages extensions. The evaluation outcome would be critical for the standardization of the wireless messages (e.g. in relevant standardization bodies such as (ETSI/ISO-CEN/SAE).

**Road infrastructure sensors**

Road infrastructure sensors are currently used to acquire traffic data (such as radar, ultrasound sensors and LIDAR) or record traffic incidents (camera). Despite the installation and maintenance costs, the data from infrastructure sensors are valuable for their reliability. However, in the future, a huge amount of data obtained from connected vehicles is expected. The connected vehicle will be able to send (and receive) real-time information to (and from) a local or central monitoring (and control) center. Connected vehicles may communicate their position, speed and other relevant information, i.e. they can act as mobile sensors. This allows for a sensible reduction (and, potentially, elimination) of the necessary number of spot sensors, which would lead to sensible reduction of the purchase, installation and maintenance cost for traffic monitoring; while, at the same time, improving the traffic estimation quality.

**Traffic Management Center (TMC)**

Nowadays, TMCs monitor traffic and provide information to vehicles, mostly related to safety. In order to incrementally move to a future where the driving manoeuvres will be controlled and the traffic mobility will be fully cooperative (Vantomme 2018), novel traffic control strategies should be involved in the TMCs activities (Iordanidou G. et al. 2016). In Section 3, several traffic scenarios and use cases, provide the potential traffic control functionalities which will be investigated within the INFRAMIX concept.

The efficiency of the traffic management is highly depended on traffic flow estimation methods for mixed traffic, comprising conventional and connected vehicles at any (even low) penetration rates. The penetration rate of
connected vehicles is a dynamic and difficult to predict factor. However, it influences the traffic estimation. This is because the estimation tools will receive information provided by connected vehicles and will fuse them with measurements stemming from a minimum number (necessary for flow observability) of spot sensor measurements; in order to deliver in real-time reliable estimates of traffic density and traffic flow by segment and even by lane, as well as travel times and incident detection.

**Vehicle and Third party services**

Two of the basic aspects in this area are the High Definition (HD) maps and the accurate localization (lane level accuracy), where different companies offer different solutions, at a limited scale though. More advanced concepts of the digital infrastructure integrate aspects of low latency communication and cloud computing; however, these are at an early stage (Lytrivis et al. 2018).

The exchange of data between an enhanced TMC as described above and traffic party services (e.g. HD map providers) will be the basis for the extraction of the in-vehicle electronic horizon and will help both automated and conventional vehicles to perform challenging maneuvers with increased safety and comfort. Currently, the electronic horizon is static and based on the digital map of the road. Learning fleet data quickly, based on a combination of data from vehicles and the infrastructure, electronic horizon could contain dynamic information about traffic flow (e.g. speed and density of vehicles, if possible in certain situations even separately for trucks and private cars) as a basis for individualized speed and lane recommendations. Such recommendations, considering traffic control strategies, will enable smoother and safer operation in dense mixed traffic, allowing for a reduction of both traffic jams and dangerous maneuvers.

After the description of the “hybrid” infrastructure concept and its components, section 3 describes the traffic scenarios, which were selected to define a set of functionalities/services of the “hybrid” infrastructure and demonstrate their impact to mixed traffic flow.

**3 TRAFFIC SCENARIOS**

The traffic scenarios under investigation are described in this section. These scenarios were carefully selected based on the following criteria:

- the expected impact on traffic flow;
- the expected impact on traffic safety;
- the importance of the challenges faced, in the sense that if not handled in a proper and timely way, they will negatively influence the introduction of AVs on the roads;
- the ability to generalize on the results (applicable in other scenarios and environments e.g. urban).

Below a high-level description of each scenario takes place, including main aspects under investigation per scenario as well as hints on the anticipated impact that will be associated with it. It should be noted that these descriptions, as well as the figures, are not detailed but indicative of the work to be performed.

**Scenario 1 – Dynamic lane assignments**

The study of this scenario intends to give us insights on how to manage at lane level mixed vehicle traffic flows on normal highway segments, that is without any tunnels, lane drops, entry or exit lanes. The purpose here is to check if a dedicated lane to AVs, either permanent or dynamic, could support AVs introduction in everyday traffic, and which are the related implications in order not to influence in a negative way current traffic.

During this process, parameters such as the penetration rate of AVs and the prevailing traffic conditions will be considered. In addition, speed limits per lane or road segment will be dynamically adapted taking into account also potential adverse weather conditions. An instance of this scenario is highlighted in Figure 2.

The goal is to provide proper indicators for activation and deactivation of lanes assigned to AVs, customized speed and lane recommendations for all vehicles on this segment based on prevailing traffic conditions and also visual and electronic ways for informing all vehicles and drivers involved. It should be noted that in this scenario the usage of physical segregation elements, such as road studs and/or solid yellow lane markings or others, for indicating a lane dedicated to automated traffic (similar to existing bus lanes) will also be investigated. Questions such as “At which penetration level of automated vehicles a dedicated lane for them will be beneficial in terms of traffic efficiency and safety?” and “What kind of physical elements will be used, according to the existing (or emerging) traffic regulations, to make the dedicated lane obvious to all traffic participants?” will be studied.
The assignment of a dedicated lane to automated traffic is expected to reduce the safety concerns around the penetration of the AVs to conventional traffic. Moreover, one of the targets of this scenario is to understand how to balance mixed traffic in order to maintain the traffic throughput at least at the same level, as in case of today’s traffic consisted of conventional vehicles only.

**Scenario 2 – Construction site / Roadworks zones**

One of the major safety hotspots, with many accidents both for vehicles and for the staff on site, are roadworks zones and construction sites. In addition, they pose significant challenges for efficient coordination of mixed vehicle traffic flows. The road infrastructure can play a key role and can help all kind of vehicles (connected, automated, conventional) to safely and efficiently pass through such areas, by providing extended information in real-time, such as updated maps (e.g. including the temporary yellow lanes illustrated in Figure 3), additional traffic signs, reference points on the spot for accurate localization for AVs, new traffic control measures etc. in the particular region. Both the physical and the digital infrastructure should be prepared to accommodate for such situations. An example of this scenario is depicted in Figure 3.

**Scenario 3 – Bottlenecks**

The scope of this scenario is to investigate real-time controllers, involving a variety of control measures, such as dynamic speed limits, merge assistance and ramp metering, to manage mixed traffic situations in front of bottlenecks of various kinds (on-ramps, off-ramps, lane drops, tunnels, bridges). The target is to avoid traffic flow degradation in these areas. An instance of this scenario is highlighted in Figure 4, where an on-ramp case is illustrated.
Several interesting problems and use cases will be investigated with respect to different types of bottlenecks, under various penetration rates of AVs. For example, in Figure 4 a platoon of AVs is blocking the vehicles entering the highway. In this case, we can study how the entering vehicles will smoothly join this platoon in case they are automated or how the vehicles forming the platoon will make space for the conventional ones to enter the highway. Proper guidance through the electronic horizon for AVs and the nomadic devices for the conventional ones, as well as visual and electronic signals need to be provided too. Innovative control measures to improve traffic efficiency and safety (e.g. avoid deadlocks) in such cases will be developed.

In this section, the three main scenarios are broken down into more specific use cases of interest. In that effort, the list of C-ITS services, considered by European Commission as highly beneficial to community was taken into account (European Commission 2016). The derived use cases attempt to cover the aspects of Day 1 C-ITS services list (which concern hazardous location notifications and signage applications) and additionally some of the Day 1.5 list (such as traffic monitoring and smart routing, which applies to highways). The idea behind Table 1 is to associate each use case with expected benefits and potential metrics for evaluation which are of interest.

Table 1. Use cases following the traffic scenarios (compiled from H2020 INFRAMIX project 2018)

<table>
<thead>
<tr>
<th>Traffic Scenarios</th>
<th>Use cases</th>
<th>Description of indicative evaluation fields</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dynamic lane assignment (incl. speed recommendations)</td>
<td>Real-time lane assignment under Dynamic Penetration Rate of AVs</td>
<td>Evaluation of the effect of the exclusive dedication of a lane to AVs. It allows the investigation of the traffic throughput based on their penetration rate, considering also the capacity of the road for conventional traffic.</td>
</tr>
<tr>
<td>Exceptional traffic situations- adverse weather conditions as an example</td>
<td></td>
<td>Taken adverse weather conditions as an example, the effect of situations that disturb the smooth operation of infrastructure services and traffic management is investigated. The maintenance of smooth traffic flow under adverse weather conditions consist an objective.</td>
</tr>
<tr>
<td>A conventional vehicle drives on a dedicated lane for AVs</td>
<td></td>
<td>Investigation of the consequences to traffic efficiency and safety, when a conventional vehicle driving on or entering a lane dedicated to AVs.</td>
</tr>
<tr>
<td>Roadworks zones</td>
<td>Single Lane Closure (e.g. short term constructions)</td>
<td>Investigation of the necessary V2X communication, visual signs as well as physical elements when a construction zone is placed in a road segment and evaluates the efficiency of that communication in the aspect of safety and user’s appreciation. The key aspect is to ensure that all kind of vehicles are timely and sufficiently informed about the roadworks zone to act accordingly.</td>
</tr>
</tbody>
</table>
Having an overview of the “hybrid” road infrastructure and its potential functionalities (sections 2 and 3 respectively), the following section contains a preliminary work on its evaluation, while providing an overview of the existing evaluation methodologies and efforts.

4 EVALUATION METHODOLOGY FOR HYBRID ROAD INFRASTRUCTURE

Over the past decade, a large number of Field Operational Tests (FOT) have been conducted in Europe, the US, Japan, Australia, and other countries to test Intelligent Transport Systems (ITS). The European Commission has sponsored several large-scale FOTs (Barnard et al. 2015). As outlined in (Barnard et al. 2016), Advanced Driver Assistance Systems (ADAS), including cooperative systems (with communication between vehicles or between vehicles and infrastructure), have been tested with thousands of drivers in real traffic conditions. Examples of these FOTs were euroFOT (Kessler et al. 2012), TeleFOT (Mononen et al. 2013), DRIVE C2X (Schulze et al. 2014) and FOTsis (Alfonso et al. 2015). The FOTs had as an objective to comprise a comprehensive program of research to assess the impacts of Information Communication Technology (ICT) systems on driver behavior, both in terms of benefits for drivers (e.g. more comfort and increased safety) and of larger scale socio-economic benefits (e.g. less congestion and fewer accidents) (Barnard & Carsten 2010). A handbook was developed with many practical recommendations by the FESTA consortium that was granted to develop a FOT methodology before large-scale FOTs would be funded (FESTA 2016; Regan & Richardson 2009). The basis of this handbook was a methodology, to be followed by the FOTs in order to ensure scientifically sound studies and allowing comparability between FOTs (Carsten & Barnard 2010). Since 2008 this methodology has not only been adopted by FOTs funded by the European Commission but also by many nationally (or otherwise) funded projects, and has influenced FOTs outside Europe. The methodology has been regularly updated by the FOT-Net support actions, taking into account the lessons-learned (www.fot-net.eu). The FESTA methodology is summarized in Figure 5. There are several steps, which although described in a linear way, are performed in iteration.

The V-shape shows the dependencies between the different steps on the left and right-hand side of the V. The steps can be summarized as:

- Defining the study: Defining functions, use cases, research questions and hypotheses
- Preparing the study: Determining performance indicators, study design, measures and sensors, and recruiting participants
- Conducting the study: Collecting data

<table>
<thead>
<tr>
<th>Step Description</th>
<th>Methodology</th>
</tr>
</thead>
<tbody>
<tr>
<td>New Lane Design (e.g. long term constructions)</td>
<td>Investigation of V2X communication, visual signs as well as physical elements in order to reassure a smooth and efficient traffic flow when roadwork zone covers more than one lane in a road segment. It is focused on the required visual signs that depict the new lane marking, the possible eHorizon applications that help an AV to accurately follow the new lane markings and the establishment of the required interface.</td>
</tr>
<tr>
<td>Bottlenecks</td>
<td>Investigation of a traffic management concept to exploit AV capabilities towards increased traffic flow efficiency by changing the AVs longitudinal driving behavior according to the traffic management requirements. More specifically, the control strategy receives real-time measurements (or estimates) of the current traffic conditions and suggests to the AVs (or to the connected conventional ones which are equipped with ACC (SAE level 2)) an appropriate value for the time-gap parameter and possibly also for the vehicle acceleration.</td>
</tr>
<tr>
<td>AVs Driving Behavior Adaptation in Real Time at Sags</td>
<td>Investigation of a traffic management concept to decide on the necessary lane-changing activities in order to achieve a pre-specified (possibly traffic-dependent) lane distribution of vehicles while approaching a bottleneck, aiming at increasing the bottleneck capacity. A control strategy is fed with real-time lane-specific information about the prevailing traffic conditions in order to provide the lane-changing recommendations.</td>
</tr>
<tr>
<td>Lane-Change Advice to connected vehicles at Bottlenecks</td>
<td>Investigation of a traffic management concept to exploit AV capabilities towards increased traffic flow efficiency by changing the AVs longitudinal driving behavior according to the traffic management requirements. More specifically, the control strategy receives real-time measurements (or estimates) of the current traffic conditions and suggests to the AVs (or to the connected conventional ones which are equipped with ACC (SAE level 2)) an appropriate value for the time-gap parameter and possibly also for the vehicle acceleration.</td>
</tr>
<tr>
<td>Lane-Change Advice combined with Flow Control at Bottlenecks for all vehicles</td>
<td>Investigation in improving the traffic flow at bottlenecks with a control of the upstream. Several innovative flow control strategies are investigated with different approaches (ramp metering, Mainstream Traffic Flow Control (MTFC)).</td>
</tr>
</tbody>
</table>
- Analyzing the data: Storing and processing the data, analyzing the data, testing hypotheses, answering research questions
- Determining the impact: Impact assessment and deployment scenarios, socio-economic cost benefits analysis

The hybrid road infrastructure examined in this paper is a relatively new research domain, for which new evaluation methods might be needed. As a first step the evaluation method that is currently utilized in the discussed study is the FESTA V-process methodology. As already mentioned in the introduction, the FESTA Handbook v.7 (2017)³, is formulated with the target to evaluate ADAS and in-vehicle information systems for vehicles through FOTs. The latest version of the handbook includes apart from in-vehicle systems also nomadic and cooperative ones, which are intended as a combination of hardware and software enabling one or more ICT functions in vehicle level.

The proposed methodology attempts to adapt FESTA, so as to be able to evaluate the so called “hybrid” infrastructure in terms of traffic safety, traffic efficiency, users’ appreciation and technical feasibility. The remainder of this section focuses on the left half of the adapted FESTA V model (Preparing).

![Figure 5. Festa V-model (source: FESTA 2016).](image)

**Step 1 Function identification and description:**

In this study, the functions to be evaluated are formulated in terms of traffic scenarios that the infrastructure should be able to handle (during the transition period, so as to become the basis for future automated transport systems).

A clear method was followed, as mentioned in section 3 (see also Figure 6), in order to select the following three key traffic scenarios:

- S1 (Scenario 1): Dynamic lane assignment (incl. speed recommendations)
- S2 (Scenario 2): Construction sites / Roadwork zones and
- S3 (Scenario 3): Bottlenecks (on-ramps, off-ramps, lane drops, tunnels, sags).

Step 2 Use Cases:

Following the identification of the scenarios and the description of the associated infrastructure functionalities, a set of use cases was developed. These use cases constitute a set of representative traffic situations for each scenario. The formulation of the use cases was performed based on the current technological level of the road infrastructure (based on the status and expert opinions of two major European infrastructure operators) and the issues that are expected to evolve within the transition period. The first use case for the Dynamic Lane Assignment scenario is provided hereafter as an example.

<table>
<thead>
<tr>
<th>ID</th>
<th>S1-DLA-UC1-DPR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Name</td>
<td>Real time lane assignment under Dynamic Penetration Rate of AVs</td>
</tr>
<tr>
<td>Overview</td>
<td>A lane is assigned dynamically to AVs in mixed traffic, when their percentage is above a certain limit, taking also into account the capacity of the road portion left for conventional traffic</td>
</tr>
</tbody>
</table>

Step 3 Research Questions and Hypotheses:

After the definition of the functions and the use cases, the research questions and hypotheses are formulated. The first step was the analysis of the original objectives and visions. This work feeds top-down the definition of the use cases, success criteria, evaluation methods and tools to evaluate the functionalities under study. These high level objectives are expressed in the form of research questions, which are grouped into the three main evaluation fields:

- The assessment of the impacts on traffic efficiency and traffic safety,
- The evaluation of the user appreciation, such as drivers/travelers and infrastructure operators,
- The evaluation of technical performance and technical feasibility of the selected road infrastructure scenarios.

For example, for the first use case (S1-DLA-UC1-DPR) the evaluation should target providing insights on the changes in traffic efficiency when a lane is dedicated to AVs under different penetration rates. It would also be meaningful to understand how the users react or if the users appreciate the infrastructure physical adaptations and investigate stakeholders’ benefits (e.g. freight companies) related to functionality of different groups of AVs using a permanently dedicated lane at different time intervals.

Following the aforementioned reasoning, a set of potential Research Questions (RQs) for the first use case (S1-DLA-UC1-DPR) could include the following:

- RQ1: At which % of automated vehicles a dedicated lane is more appropriate in terms of traffic efficiency?
- RQ2: How the throughput of conventional vehicles is affected when there is a lane assigned to AVs?
- RQ3: How does the way of providing information about a lane assignment affect the driver/passenger attitude?
- RQ4: Which is the adequate number of gantries per kilometric distance that should be installed to inform the non-connected vehicles about the dynamical lane assignment?
- RQ5: How much does the location of the dedicated lane (left or right) affect the traffic throughput?
- RQ6: Is the signaling comprehensible by the road users?
- RQ7: Do the road users appreciate the information provided during the activation/deactivation of the dedicated lane?
This approach is followed for the formulation of the research questions for all use cases that the system is intended to address. It should be noted that this process may lead to the formulation of a number of research questions. In this case a proper prioritization scheme should be applied, in order to conclude which research questions should be tested during the study. As an example, the research questions could be further prioritized based on the relevance of the assumed impact with regard to the purpose of the scenario, or for example the assumed size of the impact in the transport system.

The hypotheses translate the research question into a more specific and statistically testable statement. For example, for the first research question the formulation of the hypothesis could be the following:

- **RQ1-H1:** Traffic efficiency will increase if a dedicated lane for AV is set when AV % is over X

### Step 4 Performance Indicators

The next step to the definition of statistically testable hypotheses for the prioritized research questions is to find measurable indicators to test the hypotheses. Whereas research questions are general questions phrased as real questions ending with a question mark to be answered by compiling and testing related specific hypotheses; hypotheses are statements which can either be true or false. Hypotheses will be tested by statistical means. Defining testable hypotheses may be quite a challenge for road automation studies, as we cannot always predict what the effects are going to be. Hypotheses can only be tested by means of performance indicators (PI). So after establishing these indicators, measures need to be defined and an experimental design to be developed. A matrix was developed for this purpose, which helps to associate the use cases with the research questions, the hypotheses and the performance indicators.

Following the previous example, for RQ1-H1 the PI could be the measurement of the throughput under different penetration rates of AVs and compare it to the baseline (derived from historical data on the conventional traffic). The measures needed in order to calculate the PI are the number of AVs against the number of conventional vehicles in a highway section, logged in a traffic simulation scenario.

It should be noted that this work is in progress within the research team; in this paper we present the approach followed in order to evaluate the hybrid road infrastructure. It is acknowledged that FESTA has a strong focus on the drivers of vehicles, and the changes in their behavior when driving a vehicle that is instrumented with new systems. Here, the main target is to design, upgrade, adapt and test (in simulation and in real-world) both physical and digital elements of the road infrastructure, to enable the coexistence of automated and conventional vehicles, ensuring an uninterrupted, predictable, safe and efficient traffic. Although FESTA is focused on the vehicle functionalities evaluation, it was deemed realistic to adapt the process to a road infrastructure evaluation perspective. According to (Barnard, Y., et al. 2016) this could be considered as a context centred test, addressing questions of how mobility changes, how this affects mobility services, what the impacts are on traffic flow level or on transport system level, what ethical choices might be involved, and what would be the impacts on the built-up environment and society. These types of questions are extremely important but not easy to investigate as these impacts typically take a longer period of time to evolve than the duration of a typical test and, certainly, these changes do not take place with the penetrations that a test study is able to put up.

### 5 FUTURE WORK

As previously mentioned, the evaluation methodology described in this paper is still at its initial phase, thus further iterations and refinement is needed. FESTA methodology, on which this work is based, is well established for evaluation of ADAS functions, however further work is needed for the needs of connected and automated mobility and especially for the needs of road infrastructure and mixed vehicle traffic flows, as described in the last part of section 4. Ongoing supportive work in this field is carried out in sibling projects such as L3Pilot, which is dealing with pilot activities involving automated vehicles of SAE level 3. Several aspects on the novel automated functionalities of these vehicles (such as their operational context) and their implications on the traffic flow would give valuable information on the evaluation.

An important step towards this direction is the collection of related data and potentially big data. The more data available, the better for improving the evaluation process. Currently, the existence of AVs (of SAE level greater than 1) in the highways is very limited. Consequently, there is a lack of real data and statistics related to safety, users’

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appreciation and traffic efficiency of mixed vehicle traffic flows. This fact makes the advanced simulation in such cases an indispensable part of the evaluation of the “hybrid” infrastructure and the corresponding developments. Extended simulations will be performed, within INFRAMIX project, using an advanced simulation environment. The INFRAMIX Co-simulation environment (Lytrivis et al. 2018), combines the modelling of the vehicle behavior with the traffic simulation, thus enabling the testing of the developed traffic control algorithms:

- with increased traffic densities in exceptional conditions (e.g. bottlenecks)
- with different rates of the targeted vehicle types (conventional, automated).

The intention is to compare the data from these simulation tests with the respective real data from traffic flow with conventional vehicles. This way, the current traffic situation will be the baseline in order to demonstrate the deviation of traffic efficiency indicators with different penetration rates of AVs.

Considering the co-existence of conventional and AVs, especially in traffic scenarios such as roadworks, new safety challenges which are related to “hybrid” infrastructure design parameters (such as the latency of different networks (C-ITS G5 and LTE )) are expected. Apart from the pure simulation of the traffic scenarios, coupling virtual traffic with real world is expected to give unique and valuable data for the evaluation analysis. This will be realized though hybrid testing, which will make use of a real vehicle driving through virtual traffic. Hybrid testing permits the assessment of critical traffic situations in a safe artificial environment (Lytrivis et al. 2018).

Another important future activity, in the area of evaluation, is the appreciation of the users in the corresponding developments. In order to realize and perform the three INFRAMIX scenarios new visual and electronic signals that communicate information, issue warnings or provide guidance to all highway users (conventional and automated vehicles) need to be implemented. As a major challenge, to achieve the greatest possible impact in the transport community, regarding investments and development of new road infrastructure elements, is users’ acceptance. The last part of the communication chain are the signals with which the users have direct contact. The adequateness, comprehension as well as the social acceptance (pleasantness) of the visual signs are evaluation areas of high importance. The collection of the proper data in order to evaluate these areas is critical for the evaluation. Real users’ responses and attitude to the specific signals would be of great value.

At this point, another important future development is briefly highlighted. This is linked with evaluation, in an indirect way, however its usefulness and impact are expected to be much broader within the transport community. This is named hereafter as the “infrastructure classification scheme”. This scheme, proposed within INFRAMIX, is analogous to the different initiatives for classifying automated driving systems, ranging from “no automation” to “full automation” with the most prevalent one being the SAE taxonomy (SAE 2014). The target here is the road infrastructure. Such a classification scheme will indicate the connectivity, the provided ITS services and in general the capability to host vehicles of different levels of automation in a specific road infrastructure. The status of the digital infrastructure, such as the availability of highly accurate maps, as well as the facilities of the physical infrastructure, e.g. the lane markings condition, the availability of roadside C-ITS units, the presence of segregated or dedicated lanes and other parameters (e.g. presence of VRUs), will be taken into account in order to classify the infrastructure, matching it to a specific level of automation. This work will be accompanied by a guide of how to incrementally upgrade levels of infrastructure. This is expected to support significantly the step-wise introduction of automated driving systems, and their wide adoption.

6 CONCLUSIONS

In this paper, an overview of the challenges on the implementation and assessment of a road infrastructure capable to accommodate mixed vehicle traffic flows (co-existence of conventional and automated vehicles) was presented. This was based on the “hybrid” infrastructure concept, as conceived in the H2020 INFRAMIX project. The preliminary work on the approach to the evaluation of this concept, in terms of technical feasibility, users’ appreciation and traffic efficiency, provided important considerations and set the initial steps to a structured evaluation methodology for road infrastructure, which considers the increasing penetration of AVs in the near future.

An attempt is made to adapt the FESTA evaluation methodology, which is focused on vehicle functionalities evaluation, to a road infrastructure perspective, in order to exploit the maturity and the know-how of the existing methodology. Following the FESTA structure with slight adaptations in the different perspective, seems to be a realistic approach. Several aspects of the infrastructure have already been considered to FESTA. As mentioned in section 4, (Barnard, Y., et al. 2016) considers context centred test, addressing several questions relevant to the impacts on the
traffic flow and on the built-up environment. It is important to point out that this kind of research questions, typically require a longer period of time to evolve that the duration of a typical test, attempted to be identified also in this work along with the parameter of various AVs penetration rates.

Three traffic scenarios (dynamic lane assignment, roadworks and bottlenecks) were used as a basis for the evaluation of such concept. The three scenarios were further divided into eight use cases. These use cases were extracted taking into account the current technological level of the road infrastructure and the anticipated challenges during the transition period while targeting to demonstrate and assess the impact of the infrastructure developments. Those scenarios and use cases, although targeting highways, can provide important insights to the evaluation work, which without loss of generality can be applied also to urban environments.

Another important concept introduced in this paper is the infrastructure classification scheme. A scheme similar to SAE levels of automation for automated driving, but for the needs of the infrastructure this time. The work regarding this scheme will make road infrastructure owners and road operators to be more involved in the discussion regarding automation and be more active and supportive and in fact promoting early adoption of automated vehicles. Already several international stakeholders have expressed interest in the area of infrastructure classification and it is expected to be an important step towards a holistic automated transport system in the future.

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Managing the transportation infrastructure in the age of autonomous vehicles and intelligent infrastructure

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infrastructure, asset management, intelligent, human-centered, automated vehicle

A transportation infrastructure with high coverage and quality that provides effective and efficient mobility and access to all is critical for economic development and quality of life. However, there is a gap between user and industry expectations and the level of service provided by the transportation infrastructure. A recent study suggested that it may be possible to reduce the infrastructure gap by improving infrastructure productivity, and effective asset management is one of the tools that may help increase productivity and bridge this gap. This paper explores the impact of emerging “smart” or “intelligent” technologies on the way we manage our transportation infrastructure as we conceive, design, and construct human-centered communities and fully deploy autonomous vehicles and intelligent infrastructure.

In light of these disruptive technological changes, future asset management practices and tools are expected to (1) consider high-level goals and performance measures that reflect the emerging human-centered community paradigm and provide a way of aligning infrastructure investments with users’ and communities’ visions and goals; (2) incorporate asset condition assessment and performance monitoring indicators that reflect the needs of smart infrastructure assets and vehicles; and (3) take full advantage of the wealth of data generated by these intelligent assets and vehicles, which will likely allow transportation assets to self-monitor their condition, assess their performance, and even trigger interventions to enhance their condition and/or level or service.
Managing the Transportation Infrastructure in the Age of Autonomous Vehicles and Intelligent Infrastructure

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1 INTRODUCTION

Infrastructure is the foundation that connects the natural environment to the economy and social systems by facilitating the movement and exchange of goods, services, and people. The consequence of this connection is that the quality of infrastructure has a direct impact on the economy, the quality of the natural environment, and the quality and equity of societies (Flintsch & Brice 2014). All of these aspects are key contributors to sustainable development.

In particular, a transportation infrastructure with high coverage and quality that provides effective and efficient mobility and access to all is critical for economic development and quality of life. However, in many countries, this national asset has been neglected and there is a gap between user and industry expectations and the level of service provided by the transportation infrastructure. As the resources available for managing and expanding infrastructure are limited, there is a need for a paradigm change. A recent study suggested that it is possible to reduce the infrastructure gap by 60% by improving infrastructure productivity (McKinsey 2013). One approach to increase productivity and help bridge the gap is to improve the transportation asset management process by adopting practices that consider the whole life-cycle impacts of decisions, optimize investments, and align them with sustainable development objectives. Thus, effective asset management can be one of the tools used to improve the efficiency of the public investment in transportation and achieve needed productivity improvements.

An exogenous factor that makes infrastructure challenges even more serious is that the transportation landscape is fast changing, as many disruptive technologies are becoming mainstream, placing additional demands on our infrastructure. However, the deployment of many of these technologies can also be an opportunity, as they provide a wealth of useful information that can support decisions regarding the preservation, renewal, and expansion of our transportation infrastructure. For instance, smart cars and intelligent infrastructure assets and systems can “sense” the condition and level of service of transportation systems, communicate the information in real time to asset management systems, and provide important clues to managers about the types of interventions needed to optimize their performance.

The increasing complexity of our infrastructure networks and systems is another compounding factor. Society is supported by a broad set of interdependent networks, which include (a) physical networks such as civil infrastructure systems; (b) social networks such as communications, trade, and political systems; and (c) business environments based on information and financial interconnections (NAS 2005). Each network affects and relies on the others to provide services that society demands. In particular, efficient, well-maintained, interconnected transportation infrastructure networks play a critical role in supporting societal stability, sustainable growth, quality of life, and resilient responses to natural and man-caused disasters.

2. OBJECTIVE

The objective of this paper is to explore the impact of emerging “smart” or “intelligent” technologies on the way we manage our transportation infrastructure as we fully deploy autonomous vehicles and intelligent infrastructure and conceive, design, and construct human-centered communities.

3. THE IMPORTANCE OF MANAGING THE TRANSPORTATION INFRASTRUCTURE

Several studies have shown that the density and quality of the road infrastructure is directly related to economic development (Queiroz et al. 1994). The condition of the transportation infrastructure has an important impact on mobility and access (availability, cost, etc.) and thus is an important development engine. Furthermore, an international study (McKinsey 2013) has estimated that there is a significant worldwide infrastructure deficit and that bridging the gap at the current level of productivity would take an investment four times the level of the last 20 years.
3.1. The Transportation Asset Management Philosophy

There is an urgent need to increase infrastructure productivity, and the adoption of efficient and effective transportation asset management approaches can play a key role in this productivity increase. Furthermore, recent technological advancements, such as smart or intelligent vehicles and infrastructure assets, are creating a wealth of new challenges and opportunities related to the approaches and tools used to manage our transportation assets.

Transportation asset management has emerged as a “philosophy” that helps road agencies to:

1. Focus efforts and resources on achieving the long-term performance objectives of the agency and the country (economic, social, technical, etc.);
2. Find optimal and sustainable investment portfolios and work plans;
3. Define and implement efficient, effective, and accountable business processes for the selection, design, and execution of investments; and
4. Use technical tools that include reliable data, appropriate analytics that are aligned with the agencies’ objectives, and simple and efficient asset management systems and tools.

To facilitate the adoption of best practices, several organization have developed asset management manuals, standards, and guidelines (AASHTO 2002; ISO 2014; PIARC 2016). Among them, the manual prepared by the World Road Association (PIARC 2016) captures best practices in the road sector at the international level. This manual emphasizes the development of an asset management strategy, which documents how road asset management is delivered for the agency to meet its long-term corporate goals and objectives. It also defines three asset management maturity levels (basic, proficient, and advanced) and the business processes that define each level. In addition to having an asset management strategy, an agency with an advanced level of maturity should have a computerized asset management system and a communication strategy and have implemented the following business processes:

1. Inventory and condition assessment;
2. Performance gap analysis;
3. Life-cycle planning;
4. Risk analysis;
5. Financial plan;
6. Resource allocation and preparation of a work program;
7. Asset management plan;
8. Asset valuation; and

3.1. Transportation Asset Management Challenges

Although transportation asset management is becoming mainstream, it still faces many challenges. Chief among them is the need to manage a large number of assets under scenarios of high levels of deterioration, shrinking budgets, increasing demands, and higher user expectations. As our infrastructure networks are becoming more complex and interconnected, the traditional approach of managing these interacting systems using separate management systems that focus on particular “asset types” such as pavement or bridges is not optimal. The increased role of the private sector in the management of our infrastructure is changing the infrastructure business model and widening the spectrum of goals for the management of the assets. Finally, the data collection associated with the inventory, condition assessment, and performance monitoring of the large networks of assets is a significant and usually costly task.

It is the opinion of the authors that to advance the field, mitigate the challenges, and provide the maximum value to transportation agencies, future asset management will need to:

1. Involve low-cost, easy-to-maintain sensing and data management technologies, including integrated approaches to distribute resources within a single integrated portfolio of investments;
2. Consider the value and use of the asset in conjunction with cost of preservation and/or renewal;
3. Consider many stakeholder perspectives, values, and needs; and
4. Be robust to future ownership and usage changes. Novel solutions made possible by emerging “smart” sensing and communication technologies can play an important role in mitigating these changes.
4. IMPACT OF EMERGING TECHNOLOGIES

The world is witnessing a dramatic transformation due to a confluence of disruptive trends:

1. Rapid urbanization and explosive growth in population that is shifting the locus of economic, social, and technological activity toward emerging countries and communities;
2. Accelerating technological change leading to transformative advances in information technology, health care, manufacturing, and transportation, which is leading to instant communication and boundless data;
3. A more connected world in terms of how information, mobility, and trade are concerned, in which complex, co-evolving, socio-technical networks are enabling more efficient flow of capital, goods, people, and information.

All of these changes will have an impact in the way we conceive, design, construct, and manage our infrastructure, and in particular our transportation assets. This section discusses three technological trends that will likely impact the future of transportation asset management: human-centered communities, intelligent infrastructure, and automated vehicles.

4.1. Human-Centered Communities

The fast growth of urban population and large metropolitan areas is leading urban planners and infrastructure providers to focus on creating human-centered communities. In a human-centered community, the infrastructure networks that support the community co-evolve with the communities that use and modify them. Their development involves the human perspective in all steps of the process to optimize the economic and social impacts and minimize negative environmental impacts. Decisions and investments focus on the users’ needs and requirements. There is an emphasis on enhancing effectiveness and efficiency; improving human well-being, user satisfaction, accessibility, and sustainability; and minimizing possible adverse effects of use on human health, safety, and performance. These aims are clearly in line with the goals of sustainable development and play a role in the way we manage the infrastructure networks that support these communities.

Recent technological developments are converging to allow us to continuously maintain our social connections and responsibilities as data becomes ubiquitous and mobility ever-increasing. This is expected to transform how people conceive and create sustainable, adaptable, and resilient communities (VT 2017). However, to maximize the benefits on quality of life for current and future generations, infrastructure development and management goals, as well as associated performance measures, have to be aligned with users’ values, needs, requirements, and aspirations. Therefore, it is important that future asset management practices and tools consider this emerging paradigm, as it will provide a way of aligning the infrastructure investments with the community’s vision and goals.

4.2. Intelligent Infrastructure

Intelligent or smart infrastructure technologies are also likely to significantly impact the way we manage our transportation infrastructure assets. Sensing and communication technologies are making our infrastructure assets “smart,” allowing them to self-monitor their condition and performance and even trigger interventions to enhance their condition of level or service. For example, a recent report (CISE 2017) defined an intelligent infrastructure asset as one that “is linked to information and rules governing the way it is intended to be constructed, maintained, used, refurbished and demolished and enable the asset to support or influence its own use.”

There are a large number of sensors that are being developed that can provide useful information about the condition and performance of key infrastructure assets and support the allocation of the reduced budgets available for maintaining these assets. These sensors are being deployed around the world, allowing key asset performance indicators to be monitored in real time, and can be used in future asset management to directly, and possibly automatically, trigger actions based on the information provided by the assets themselves.

Instrumented infrastructure brings to the asset management world the notion of self-contained assets linked to their own monitoring, diagnostic, and maintenance strategy that have the ability to guide, influence, or direct their own use, maintenance, and support. These intelligent infrastructure assets are easy to identify and locate, self-monitor their condition, automatically assess the level of service provided, and communicate the data to be used to support, and even trigger, asset management decisions (CISE 2017). This automatic and continuous monitoring can reduce the risk that asset management operators might miss key indicators or fail to schedule critical maintenance tasks.

However, the widespread deployment of these technologies will also create additional challenges for asset management as it will change the way we collect, process, manage, analyze, and interpret the enormous amount of data that is generated by intelligent assets. Furthermore, the possible automation of some of the maintenance decisions would have significant organization and legal implications.
4.3. Automated Vehicles

Over the last decade, cars, trucks, and buses have become increasingly “smarter” as they incorporate many safety technologies that help the driver keep control in difficult situations and avoid or reduce the severity of crashes. SAE (2017) has identified six levels of driving automation that provide a continuous spectrum from “no automation” to “full automation” that provide a framework for a stepwise progression toward full automation.

At the lowest levels of automation (levels 0 and 1), technologies such as collision prevention and mitigation systems (e.g., forward collision warning, brake assist, and pedestrian detection), lane departure warning, electronic stability control, adaptive lighting, and adaptive cruise control are playing an important role in reducing crashes and associated fatalities on roads. These technologies use sensors that in many cases rely on a transportation infrastructure that is robust and “predictable.” For example, most available lane departure warning and “autopilot” systems rely on pavement markings for lateral vehicle positioning; thus, it is important that the asset management process places a high priority on providing adequate markings and includes performance measured based on retro-reflectivity measurements and intervention thresholds for replacing the markings.

On the other hand, vehicle sensors offer a great opportunity to improve asset management, as the data collected can be used to assess the condition and level of service of the road infrastructure. For example, average vertical accelerations can provide a general indicator of serviceability in terms of ride quality, and frequent activation of vehicles’ electronic stability control systems can pinpoint potentially hazardous slippery pavement sections (Flintsch et al. 2012; Katicha et al. 2015).

Vehicle connectivity is also becoming increasingly common to enhance safety and capacity. Many of the new vehicles incorporate technology to communicate with other vehicles (V2V) and with the infrastructure (V2I), which is used mainly for informing drivers of potential hazards but can also support the management of transportation systems. For example, information collected from connected “smart” vehicles is already being used in some Scandinavian countries to inform and support winter maintenance operations.

At the high end of the spectrum are fully automated vehicles, in which there is “full-time performance by an automated driving system of all aspects of the dynamic driving task under all roadway and environmental conditions that can be managed by a human driver” (SAE 2014). These systems are still experimental and are expected to rely on both sensor and communication technology. The condition and level of service of the infrastructure, and consequently the way we manage it, would be especially critical for this type of vehicle as unexpected events, such as potholes or sudden changes in pavement properties, may create severe disruption of the automated driving system.

4. CONCLUSIONS

Effective asset management can help improve the efficiency of public investments in transportation and achieve some of the productivity improvements needed to bridge the infrastructure gap that is hindering development and negatively impacting quality of life worldwide. However, there are still many challenges associated with the full deployment of transportation asset management.

Disruptive “smart” technologies are becoming mainstream and impacting the way we conceive, design, construct, and manage our infrastructure, and in particular our transportation assets. These changes are placing additional demands on our transportation infrastructure and challenging the way we manage it. Challenges include the definition of performance standards that are compatible with emerging vehicle technologies and the development of human-centered communities, and the collection of the needed condition and level of service data for a large number of complex and interconnected infrastructure assets and systems. However, the deployment of smart vehicles and infrastructure assets also provides opportunities as they can provide a wealth of useful information that can help support decisions regarding the preservation, renewal, and expansion of our transportation infrastructure.

In light of these disruptive technological changes, future asset management practices and tools are expected to evolve to:

1. Consider high-level goals and performance measures that reflect the emerging human-centered community paradigm and provide a way of aligning infrastructure investments with users and communities’ visions and goals;

2. Incorporate asset condition assessment and performance monitoring indicators that reflect the needs of smart infrastructure assets and vehicles; and
(3) Take full advantage of the wealth of data generated by these intelligent assets and vehicles, which will likely allow transportation assets to self-monitor their condition, assess their performance, and even trigger interventions to enhance their condition and/or level or service.

5. REFERENCES


