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IRF GLOBAL R2T CONFERENCE & EXPO: Technical & Scientific Papers

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EDITOR
Sam L. Enmon II

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**ABSTRACT:**

The Izmit Bay Suspension Bridge in Turkey is heart of the Gebze-Orhangazi-Izmir motorway BOT Project. The bridge, contracted to IHI Infrastructure Systems Co., Ltd. on an EPC basis was constructed with a 1550m main span making it the world’s fourth longest suspension bridge. The scale of the bridge and the tight schedule in the EPC contract required state of the art, suitable design, well-proven construction methods adapted to the technical challenges and financial success for the project.

The tower foundations are concrete caissons placed 40 m below sea level on a gravel bed over soil. The tower foundation is allowed to move by releasing friction, functioning as an isolation system under strong earthquake.

The deck is divided into 117 segments of approx. 25m length. The segments size was decided by the capacity of lifting devices.

The two towers are 252 m high, each consisting of two single-celled box legs with two cross beams. The overall dimensions of the cells are 7m x 8m at the base and taper to 7m x 7m at the top. Each tower leg is divided into 22 blocks and connected by welding for the perimeter plates and by bolting for the vertical stiffeners.

The main cables are made of pre-fabricated parallel wire strands, each consisting of 127 high strength wires of 5.91mm in diameter and having a breaking strength of 1760 MPa.

Construction was completed in 3.5 years, and the bridge opened to traffic on July 2016. This paper deals with the project overview including design, construction, and schedule.
Izmit Bay Suspension Bridge  
Project Overview, Construction and Schedule

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1 INTRODUCTION

The Izmit Bay Suspension Bridge carries the new Gebze-Orhangazi-Izmir motorway across the Sea of Marmara at the Bay of Izmit in northern Turkey as shown in Figure 1.

The new Gebze-Orhangazi-Izmir motorway was contracted between OTOYOL YATIRIM VE ISLETME A.S formed by Nurol, Ozaltin, Makyol, Astaldi, and Gocay (NOMAYG) and the General Directorate of Highways, Turkey (KGM) as a Build-Operate-Transfer (BOT) project for 22 years and 4 months in September, 2010. The NOMAYG joint venture formed by the same five companies as for OTOYOL, was designated as the single EPC implementing body to construct a 420 kilometer road including the Izmit Bay Suspension Bridge. The Izmit Bay Suspension Bridge was contracted to the IHI Infrastructure Systems (IHI) in July 2011 by NOMAYG.

The scope of IHI’s works is Engineering-Procurement-Construction (EPC) of the IZMIT Bay Suspension Bridge including transition spans at both ends. Under the IHI’s scope, several subcontracts were signed for specialist works with unique entities, such as Detailed design for COWI (Denmark), Independent design checker (IDC) for Halcrow (UK) and TYLI(USA), Soil investigation for Fugro (USA), Substructure works for STFA (Turkey), Cable manufacturing for Tokyo Rope (Japan), Cable saddle manufacturing for Cividale (Italy), Steel fabrication of Tower and suspended Deck for Cimtas (Turkey), Steel shaft for TGE (Turkey), Transition deck for Cimolai (Italy), Electrical products for Siemens (Turkey), etc.

Construction of the Izmit Bay Suspension Bridge was started in 2011 with the design and preparatory works, moving to major construction works on site in January 2013. Construction was completed for opening the traffic on June 2017.

2 DETAILED DESIGN

The EPC Contract to start the detailed design was awarded by NOMAYG in September, 2011. The detailed design of the Izmit Bay Suspension Bridge was carried out by the designer COWI in Denmark under supervision by IHI for the basis study, including the wind tunnel test, geotechnical investigations and geotechnical analysis. All deliverables in the detailed design were endorsed by Halcrow in UK as the IDC consultant, and main items in design were completed as of the end of February, 2013.

2.1 GEOTECHNICAL INVESTIGATION

In order to collect the necessary information to finalize the locations of foundations and to establish the design basis for the foundation design, the geotechnical investigation, which includes (1) the offshore geotechnical investigation for the tower foundation area, (2) the near shore technical investigation for the south anchorage area, and (3) onshore drilling for the north anchorage area was conducted by Fugro.
in 2011 under a contract from and supervision by IHI. One important discovery during soil investigation was the existence of a potential secondary fault far below the ground level but at close location to the south anchorage as shown by red line in Figure 2. This resulted in shifting the south anchorage northwards by 118m, shortening the side span length while maintaining the main span length, and changing the design concept of south anchorage as stated in section 3.1 of this paper.

2.2 SEISMIC DESIGN

In 1999, an earthquake of magnitude 7.6 (Richter scale) earthquake hit Izmit City which is about 30km away from the bridge location along the western portion of the North Anatolian Fault Zone. Due to this active fault zone, 3 levels of ground motion have been considered in seismic design of this bridge: no collapse earthquake (NCE), safety evaluation earthquake (SEE) and functionality evaluation earthquake (FEE) which corresponds to 2475, 1000 and 150 years of return period respectively. The spectra acceleration for each ground motion is shown in Figure-3.

2.2 WIND TUNNEL TEST

The fundamental basic wind velocity is taken as 25.4 m/s at 10m above the ground, based on the measured wind data at three meteorological stations near the construction site. The critical wind velocity for instability is 58 m/s for the horizontal wind.

In order to ensure aerodynamic stability for the deck section, particularly against flutter instability, preliminary wind tunnel tests using sectional models of a scale of 1:40 and 1:48 were conducted at BMT in UK in 2011. A series of wind tunnel tests of the deck were conducted for several deck sections of different shapes (mono box and twin box) and different depths. A 4.75 m deep streamlined line mono box girder with inspection walkway on each side was found most suitable, and chosen to be used for the detailed design. The deck sectional model of a geometric scale of 1:65 was tested in smooth and turbulent flow in FORCE in Denmark to check the stability, and to collect all necessary data, for construction and in-service condition.

The tower aerodynamic model of a geometric scale of 1:220 was tested in smooth and turbulent flow in BLWT in Canada to investigate aerodynamic stability and derive static load coefficients for construction and in-service conditions.

The full bridge model of a geometric scale of 1:220 was tested in smooth and turbulent flow in Politecnico di Milano in Italy finally to confirm aerodynamic stability of the suspension bridge during construction and in-service condition as shown in Figure 4.

3 BRIDGE CONFIGURATION AND PROFILE

The final bridge configuration is shown in Figure 5. The main span is 1550m and the side span is each 566m. The main span of 1550m is the world’s fourth longest suspension bridge at completion in 2016. The suspended deck is 2682 m long and continuous between two side span piers. The main cables are deviated at the side span piers toward the cable anchorages located below the deck of the transition spans. The anchor spans of the main cable between the side span pier and the splay saddle are 92.05 m and 67.25 m for the north and the south respectively. There are 120 m and 105 m transition decks on the north and south approach respectively. The profile of bridge and the summary of quantities are shown in Table-1 and 2.

![Figure 5 Bridge Profile](image-url)
### Table 1 Bridge Profile

<table>
<thead>
<tr>
<th>Steel Works</th>
<th>Weight</th>
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<tr>
<td>Deck Weight</td>
<td>33,000 ton</td>
</tr>
<tr>
<td>Tower (North)</td>
<td>9,000 ton</td>
</tr>
<tr>
<td>Tower (South)</td>
<td>9,000 ton</td>
</tr>
<tr>
<td>Cable</td>
<td>18,800 ton</td>
</tr>
<tr>
<td>Hanger</td>
<td>800 ton</td>
</tr>
<tr>
<td>Pile</td>
<td>13,000 ton</td>
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<td><strong>TOTAL</strong></td>
<td><strong>83,600 ton</strong></td>
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<th>CONCRETE</th>
<th>REINFORCEMENT</th>
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<tr>
<td>Anchorage (North)</td>
<td>36,000 m³</td>
<td>36,000 m³</td>
<td>5000 ton</td>
</tr>
<tr>
<td>Anchorage (South)</td>
<td>89,000 m³</td>
<td>85,000 m³</td>
<td>11000 ton</td>
</tr>
<tr>
<td>Caisson (North)</td>
<td>25,000 m³</td>
<td>21,000 m³</td>
<td>4500.00 ton</td>
</tr>
<tr>
<td>Caisson (South)</td>
<td>50,000 m³</td>
<td>21,000 m³</td>
<td>4500 ton</td>
</tr>
<tr>
<td>Diaphragm Wall</td>
<td>- m³</td>
<td>20,000 m³</td>
<td>2400 ton</td>
</tr>
<tr>
<td>Pile</td>
<td>- m³</td>
<td>600 m³</td>
<td>200 ton</td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td><strong>200,000 m³</strong></td>
<td><strong>183,600 m³</strong></td>
<td><strong>27,600 ton</strong></td>
</tr>
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### 3.1 SUBSTRUCTURE

The design of the south anchorage has been changed much from the tender design because of the secondary fault. The separate concrete slabs in the front and in the rear in the tender design has been integrated into one large slab, on which the side span pier, the cable anchorage of triangle shape and the transition pier are placed as the anchorage as shown in Figure 6. This configuration makes the deviation saddle on the side span pier and the cable anchorage to move together under strong earthquake without creating enormous forces in the main cables.

The north anchorage is designed as a gravity type consisting of the cable anchorage triangle shape and the transition pier. These two structures are placed on the united anchor block which is directly sitting on rock. The side Span pier, on which deviation saddles are placed, is independently standing on the footing with piles as shown in Figure 7.

The tower foundations are designed as concrete caissons placed 40 m below sea level as shown in Figure 8. The concrete caisson is placed on a gravel bed over soil strengthened by steel inclusion piles (D=2m, L=34.3m, 195 nos. at each location). The tower foundation is allowed to move, by releasing friction at NCE earthquake, relative to the soil below the concrete caisson under strong earthquakes. This prevents transfer of huge seismic forces into the superstructure.

### 3.2 SUPERSTRUCTURE

The suspended deck is an orthotropic stiffened steel box girder of 4.75 m deep and 30.1 m wide having cantilevered inspection walkways of 2.75 m on each side as shown in Figure 9. The suspended deck with three traffic lanes in each direction are suspended by hanger ropes spaced at typically 25 m.

The 2682m deck is divided into 113 standard segments of 300 tons each and 25m length, and 4 special segments. The length of segments was decided by the capacity of lifting devices and hanger spacing. Before assembling the final segments in box shape by welding, 3246 individual panels were delivered to assembly shop.
The thickness of the steel deck is 14 mm for all traffic lanes, considering heavy traffic volume and more number of five axles articulated vehicles on middle and fast lanes. The thickness of bottom plates is 9mm in standard segments.

The towers are 252 m high single-cell box steel structures consisting of 4 panels and having two cross beams connecting two tower legs, separated by 30.1 m at the top and 36.2 m at the bottom as shown in Figure 9.1. The overall dimensions of the tower legs are 7m x 8m at the base and 7m x 7m at the top as tapered in the longitudinal direction. Each tower leg is divided into 22 blocks which are connected by welding for the perimeter plates and by high strength friction bolting for the vertical stiffeners. Bottom of 11 blocks are fabricated and delivered as whole boxes weighing approx. 300 tons and were erected by floating crane (FC) with capacity of 1200 tons. The remaining 11 blocks are fabricated and delivered as 4 individual panels per block, weighing approx. 40 tons/panel, and were erected by Tower crane and bolted in longitudinal joints on site. (Figure 10 and 11)

The deck, towers and cables are protected by dehumidification systems against corrosion for the entire length.

The main cables are designed as being constructed by means of pre-fabricated parallel wire strand (PPWS), each consisting of 127 high strength steel wires of 5.91mm in diameter and having a breaking strength of 1760 MPa. Per one main cable, 110 PPWS are placed between the cable anchorages and 2 extra PPWS are placed between the tower and the cable anchorage on both sides (Figure 12) The hanger ropes are designed as parallel wire strand (PWS) coated by HDPE sheath and terminated the sockets that are pin connected to the cable clamp and the hanger anchorage at the deck.
4 CONSTRUCTION PROGRAM

The project schedule as shown in the Figure 13. In the Suspension bridge EPC contract, substantial completion (traffic open) was agreed as 37 month after commencement of works which is a very challenging schedule compared with similar size of suspension bridges.

To realize this target, all main activities from Day-1 are executed by 2-shifts to minimize the duration of each activity, and to spread the works and/or resources by which works can be performed at same time. As a result, many activities are on the critical path, and worked simultaneously. There are 4 major critical activities, which are North Anchorage, South Anchorage, Tower Construction (consisting of Caisson, Tower Foundation, Tower fabrication, Tower Erection) and Cable manufacturing followed by Cable Works. After cable works, deck erection and ancillaries will be executed as series on the critical path.

And one of the advantages of this project is “quick decision”. As for the ordinal project, sometimes an immediate Engineering decision is required during the execution of works. Due to the character of the EPC contract, a designer who is under the EPC contract, is fully involved in the project and makes quick decisions appropriately. (On the other hand, Independent Design Checker is working for the security and/or confirmation of the Engineering decision for the main parts of the design, including some site decisions.) There was an accident during Catwalk installation activities, and due to this accident, subsequent activities were suspended for 5 months. Except for this event, all activities were executed as per the planned schedule.

5 CRITICAL ACTIVITIES AND PRODUCTION

After securing the project from a financial point of view, all activities were fully started to reach the goal of the project, which is to complete the bridge construction within 37 months or earlier. The critical activities and its volume of main production are stated below.
5.1 North Anchorage Construction
There are following activities for North anchorage construction as shown in Photos 1.

**Excavation:**
After Jet grouting surrounding anchorage, surface soft soil was excavated by excavator approx. 60cm, and up to -17m Hard Rock was excavated by drilling, blasting and giant breaking by each 3m layer. During excavation, large amounts of groundwater flooded the hole and disturbed this activity for a few months.

**Casting structural concrete:**
Massive concrete for footing and triangle superstructure is constructed by RC and Prestressed Concrete. Concrete was supplied from an existing commercial plant near the site. Approx. 6000m³/month of concrete was cast as standard speed for 6 months, and 8500m³ of concrete was cast at the peak month.

5.2 South Anchorage Construction
There are following activities for South anchorage construction as shown in Photos 2.

**Reclamation:**
Original location of south anchorage is shallow water. It was reclaimed up to +1.5m using excavated soil from the Dry dock and other sources.

**Diaphragm Wall:**
The shape of the diaphragm wall is dual circular to expect arch action and minimize the strut and soil anchor to stable during excavation.

**Excavation:**
Up to -15m, Soft Soil was excavated by excavator.

**Casting structural concrete:**
Mass concrete was cast in 10 layers for the footing, and triangle superstructure is constructed by RC and Prestressed Concrete. Concrete was supplied from the new plant which was constructed for this project exclusively on site. Approx. 9600m³/month of concrete was cast as standard speed for 8 months, and 12500m³ of concrete was cast at the peak month.

5.3 Caisson Construction
The following activities for caisson construction are shown in Photos 3. Approx. 2500m³/month of concrete was cast as standard speed at Dry Dock and Wet Dock, and approx. 4000m³/month of concrete was cast for plinth and tie beams.

**Works at Dry dock:**
A temporary dry dock was made before official commencement of works, and casting concrete for the lower part of Caisson (Nearby 3/4 of caisson concrete was casted) was executed after Commencement.
Towing to Wet dock:
Towing caissons from dry dock to wet dock via dredged channel.

Works at Wet dock:
Casting remaining concrete on floating caisson, and placing two steel shafts on each caisson. Steel shaft was fabricated at shipyard close to the site, and delivered to wet dock by floating crane.

Sinking at tower location:
Sinking caisson filled by sea water at tower location with a 200mm tolerance on location deviation.

Plinth & Tie beam:
After sinking operation, concrete for plinth and tie beam at top of steel shaft were casted with anchor frame of steel.

5.4 Tower Foundation
Depth of sea at Tower foundation is approx. -40m. Before receiving Caisson for Tower, the following activities are executed at each location of sea bed.

Dredging at Tower:
3m of surface material at sea bed was dredged by Grab dredger.

Inclusion Pile:
34m length, 196 nos. at each location, and 2m dia. Inclusion piles are driven by vibration hammer.

Gravel setting:
Gravel was dumped using guide pipe, and made flat within 30cm tolerance by levelling frame.

5.5 Fabrication Works
There are following activities for fabrication works as shown in Photos-5. The steel fabrication of the tower and the deck are subcontracted to local fabricator under IHI’s supervision. The tower and the deck panels were fabricated at designated shops before being delivered and assembled to erection unit (36m wide and 25m long) at selected ship yards in proximity of the erection site. The main structural steel material applied is S460 for the tower and S355 for the deck. Both were imported from EU countries.

Panel Fabrication:
After receiving material, it was cut and welded to make thousands of panels. Since there is huge amount of welding requiring high quality control, Robot or welding machines were developed and used. In addition, there is a lot of welding inspection in this stage by the certified third parties following specification. The tower was executed as class EXC-3 and girder was executed as class EXC-4 at initial stage and reduced to class EXC-3 for following stage.

Block assembly:
Before delivery of the site, the panels are assembled to the box by welding or temporary bolt. Accuracy of the box shapes are directly reflecting to the final quality (shape and dimension) of the products at site. Therefore there is some QC/QA measurement in this stage.

Trial Assembly:
After delivery to the site of the products, it is very difficult to repair any non-conformance in fabrication. Therefore final checking was be executed in shop by trial assembling all products. Matching of block connections were especially carefully checked and machined pieces which would serve as guides/ references of site assembly were attached at this stage.
5.5 ERECTION WORKS
There are following activities for erection works as shown in Photos-6. The steel erection of the tower and the deck were directly executed by IHI’s engineers and Japanese supervisors and Turkish workers. After the erection, the blocks were connected by welding or high strength bolts.

**Erection (Tower):**
Bottom blocks up to 11th layer of 22 layers were erected by floating crane, and remaining area from 12th to 22nd were erected panel by panel by tower crane. Regardless of bottom box erection or top panel erection, all blocks were erected by temporary bolting of longitudinal ribs without completion of welding of primary plate up to some level. After the erection transverse (horizontal) connection of outer plate were welded, and inside stiffener plates were bolted at site.

**Erection (Deck):**
17 numbers of the special blocks out of 113 blocks, such as the blocks nearby the tower, middle span, close to land, were erected by floating crane. And 44 blocks out of 113 blocks were erected by standards length which is 25m in longitudinally. And remaining 52 numbers of the blocks, out of 113 blocks, were jointed to 1 mega block by two standard blocks on land, so 52 standard /2=26 mega blocks which has 50m length were erected as mega block.

5.6 MAIN CABLE WORKS
The following activities for Cable works are shown in Photos 7. The PPWS cable manufacturing was subcontracted to Tokyo Rope Co. in Japan. Site activities were directly executed by IHI since it is a unique activity and required experienced supervision.

**Manufacturing:**
Cable rod was manufactured in Japan and delivered to China. In China, it was drawn, galvanized, bundled, and socketed in China. Finally PPWS was delivered to Turkey in three ships.

**Preparation at site:**
One of the essential events at site is crossing of Izmit Bay by hauling rope on Feb, 2015. After this event, and before erection of main cable, many activities were executed, such as Saddle installation, Hauling system establishment, Catwalk installation, Set back of Tower, etc..

**PPWS erection:**
110 number of PPWS in entirely length of bridge, and additional 2 PPWS for side span were erection by using hauling system.
6 CONCLUSIONS

This paper presented the activities, volume of works and schedule in the construction of Izmit Bay Suspension Bridge. Though these items are affected by local conditions such as seismic hazards/ environmental/ traffic requirement and working hours/ experiences/ local resources and might require adaptation for each project, the authors hope the information provided in this paper will help in the planning of future Suspension Bridge Projects.

7 ACKNOWLEDGEMENTS

The authors are deeply grateful to KGM (General Directorate of Highway, MOT Turkey) and OTOYOL YATIRIM VE ISLETME A.S for guiding us to a successful completion and their permission on publication of this paper.

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8th New York City Bridge Conference, August 25th and 26th, 2015 - Izmit Bay Suspension Bridge – Project overall and schedule M.Yanagihara and Others
SUBMISSION ID NUMBER 81

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<td>VIKRAM NALAWADE</td>
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</table>

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KEYWORDS: Expressway, launching, superstructure, resource management, Ganga bridge

ABSTRACT:
The continuity and commissioning of road projects largely depends on completion of major river bridges involved in the project. The construction of 750-meter-long Ganga bridge was an important link for early commissioning of Agra Lucknow Expressway Project. Considering revised dead line for commissioning the entire project, completion of Ganga Bridge was on a critical path and the construction time had to be compressed by 25% of the initial duration considered during tender stage. The initial plan was to complete the erection using Launching Girder (LG) system. In order to expedite the project, multiple erection systems were deployed such that parallel activities could be done simultaneously. The superstructure which was originally segmental box girders was changed to a combination of segmental and composite spans. High emphasis was given on resource management and planning in close coordination with design team to finalize the entire construction methodology. The proactive thinking initiated by the team involved in the project helped the completion of bridge superstructure in 18 months instead of 24 months.
1. INTRODUCTION

In today’s competitive world where projects are taken at Competitive margin, it is important to complete the projects on or ahead of scheduled time, to control the overheads and improve the profit margins. Early completion of projects will not only help improve the profit margins but also help the nation at large to improve economic growth and help the government to plan further development programs. Afcon has always been striving to complete the projects ahead of time by using techniques for proper planning, improving productivity, developing communication channels, engaging with stake holders to eliminate procedural hindrances and value engineering etc. All organizations plan; the only difference is in their approach. Prior to starting a new strategic planning process, it will be necessary to assess the past planning approach that has been used within the organization and determine how the organization's culture may have been affected. Addressing these cultural issues is critical to the success of the current planning process.

Afcon executed Agra Lucknow Expressway Project (ALEP), connects the heartland of Uttar Pradesh with key business centers in northern India. The 302 Km long Agra Lucknow Expressway is India’s longest access controlled expressway. Afcon has constructed 126 KM of total length, divided into two packages between Kannauj to Unnao (Package IV) and Firozabad to Etawah (Package II), approximately 42% of the total length. The expressway is constructed for a design speed of 120 Km/ph. The project is constructed as six lane expressways with eight lane configuration for all structures. The project under discussion is Development of Kannauj to Unnao (Ch. 172+500 to 236+500), access controlled expressway (Greenfield) project (Package IV) in the state of Uttar Pradesh. Amongst numerous challenges in the project was the construction of a 750 m long eight lane bridge over river Ganga. The ALEP project details are in table 1.

<table>
<thead>
<tr>
<th>Table 1 ALEP (Package IV) project details</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Project</strong></td>
</tr>
<tr>
<td>ALEP PROJECT</td>
</tr>
</tbody>
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2. CONSTRUCTION OF SUPERSTRUCTURE OVER GANNA BRIDGE

Faster construction of 8 lane Expressway Bridge over Ganga has made the project to move closer to completion. Initially the construction was planned to be completed in 24 months but considering the pace of the project the time has been reduced to 18 months. To achieve the construction of Ganga Bridge in 18 months a proactive thinking was must. The well foundation was completed in 12 months as planned without facing much difficulty in sinking and other activities. The piers of height 8.5m were constructed in single lifts (Fig. 1) using the customized shutters designed by Formpro Engineers Pvt. Ltd and fabricated in-house for the project. The customized shutters helped in completing the works on fast track.

Proactive planning is an initiative to plan in advance, schedule important events and prepare for success. The Construction of 8 lane bridge over Ganga river at Km 219+125 to Km 219+875 of length 750m with 15 spans each of length 50m was planned in the project. The erection of superstructure was on critical path. Initially the erection was planned using one Launching Girder (LG). But it was anticipated that there will be delay in fabrication of Launching Girder, which will affect the erection process and finally the completion of project. A proactive manager who applies the principle of thinking ahead recognizes needs of his organization or department and finds the resources necessary to meet them. This initiated the need to revise the erection methodology to maintain the pace of the project and look for multiple methods of erection in single bridge construction. After analyzing the site conditions and discussions it was understood that LHS can be completed using the Launching Girder and for RHS a separate scheme of erection was suggested that is ground supported staging system (GSS). Since erection could not be done with GSS alone in RHS due to perennial river. Thus, a combination of ground supported staging system and composite slabs was adopted in the RHS.
for erection of superstructure. The first ten spans were erected using GSS and balance five spans at Agra end were done using the composite slab.

3. PROCESS OF GSS
The construction of ground supported staging system (GSS) is used for erection of superstructure in the construction of Ganga Bridge from abutment A2 to P5 in RHS side as shown in figure 2. Since the GSS is required to be erected on sandy strata, the driving of temporary piles was suggested for foundation.

Due to the river bed in Ganga it has been decided to adopt the method of liner driving for erecting the GSS. Driving liners of 1200mm /1000mm diameter of 10mm thick and up to 19m in depth from the existing ground
level in three groups with four numbers in each group were planned in the project. Seating beam made of 600mm ISMB has been welded into the liner to place the single main girder and double main girder with one and two number 600mm ISMB respectively. Connecting both the liners in transverse direction the double main girder is for resting of the bottom frame and single main girder is for levelling of the middle tower by hydraulic jacks. Then over the temporary piles the structure of GSS was erected as shown in figure 3 the general arrangement drawing. It consists of two planar steel frames at the pier locations which are tied to the piers and the central towers to support the steel grid beams. The beams are supported on the towers with a transverse steel beam suspended from the bracket on the columns, using high tensile (HT) threaded rods and the hollow jacks. Above the steel grid beams, segment trolleys are placed, which support the segments with jacks as shown in figure 4.

![Figure 4: Top view of GSS arrangement ready for receiving segments.](image)

The GSS is erected in the span where segments are to be erected as shown in figure 4. The segments will be lifted by crane and placed on GSS starting from abutment to successive pier as shown in figure 5. After placing all the segments on GSS, dry matching is done as shown in figure 6. Glue is a chemical product in base and hardener form used as an adhesive and filler material which has a pot life of 60 mins after mixing. Quick Mast BSA3 of Don chemical is used in the project. Four Kg of base and two Kg of hardener is mixed to form 6Kg of glue. The glue must be applied within 60min and temporary stressing shall be done. After 60 minutes the glue loses its properties and cannot be used as adhesive or filler. The consumption of glue is 3kg/sqm and approximately one metric ton per span is observed in the project. Once dry matching is found satisfactory gluing and temporary stressing were carried out as shown in figure 7. Then the permanent stressing for the span is done after finishing the temporary stressing. Once permanent stressing completes the spans were lowered on the bearing and erection is completed for the span as shown in figure 9. The erection of GSS in second span was improved by converting some of the activities like liner driving, middle tower
erection parallelly as shown in figure 8. The last segment using the GSS was erected with great accuracy and effort as there was very less room available for the movement of the segment as shown in figure 10.

Figure 6: Dry matching in the first span (A2-P14) segments.

Figure 7: Temporary stressing after gluing of segments.

Figure 8: Erection of middle tower and liner driving in the second span.
3.1 Time cycle achieved using GSS

The time cycle achieved for the erection of box segments using GSS at ALEP project in Ganga bridge was shown in the Table 2. Initially the time taken to erect in the first span was 18 days which has been reduced to 11 days for second span and further reduced to 6 days, 5 days & 4 days in completing the erection using GSS.

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>Span</th>
<th>Segment erection</th>
<th>Alignment, dry match gluing &amp; Temporary stressing</th>
<th>Threading (Cable Pushing)</th>
<th>Final Stressing</th>
<th>Total Days</th>
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<tr>
<td>1</td>
<td>A2 - P14</td>
<td>8.0</td>
<td>5.0</td>
<td>3.0</td>
<td>2.0</td>
<td>18.0</td>
</tr>
<tr>
<td>2</td>
<td>P14 - P13</td>
<td>4.5</td>
<td>3.0</td>
<td>2.0</td>
<td>1.5</td>
<td>11.0</td>
</tr>
<tr>
<td>3</td>
<td>P13 - P12</td>
<td>3.0</td>
<td>1.0</td>
<td>1.5</td>
<td>1.5</td>
<td>7.0</td>
</tr>
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<td>4</td>
<td>P12 - P11</td>
<td>2.0</td>
<td>1.0</td>
<td>1.5</td>
<td>1.0</td>
<td>6.0</td>
</tr>
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<td>P11 - P10</td>
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<td>0.5</td>
<td>1.5</td>
<td>1.0</td>
<td>5.0</td>
</tr>
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<td>6</td>
<td>P10 - P9</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>4.0</td>
</tr>
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<td>7</td>
<td>P9 - P8</td>
<td>1.0</td>
<td>1.0</td>
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<td>4.0</td>
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<td>8</td>
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<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>4.0</td>
</tr>
<tr>
<td>9</td>
<td>P7 - P6</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>4.0</td>
</tr>
<tr>
<td>10</td>
<td>P6 - P5</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>4.0</td>
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<table>
<thead>
<tr>
<th></th>
<th>Total Time taken for erection of ten spans</th>
<th>Days</th>
<th>Minimum Time Cycle per span</th>
<th>Days</th>
<th>Average Time Cycle per span excluding the learning curve</th>
<th>Days</th>
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<td>Total Days</td>
<td>67.00</td>
<td></td>
<td>4.00</td>
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<td>4.75</td>
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It is observed that there is an initial phase of learning in the process of erection using GSS which has resulted in more time. From the third span, onwards the time cycle achieved was reduced to an average of 4 days per span. This reduction in time cycle is achieved as some of the activities were done parallel to save time. For example, the erection of middle frame and liner driving is taken up simultaneously in the second span to optimize the time cycle as shown in figure 8. The average time cycle achieved per span in the project was 6.7 days which is at par considering good engineering practices. The average time cycle achieved excluding the learning curve of two spans initially erected using GSS is 4.75 days for each span. This demonstrates a saving in time of approximately 25% in the project considering the average time taken as 6 days per span globally.

4. PROCESS OF LG

Launching Girder is used for erection of box segments in Ganga Bridge from pier P14-A2 to A1-P1 LHS side. The details of the span and other arrangements are as given in Table 2 below for reference.

<table>
<thead>
<tr>
<th>S. No.</th>
<th>Item</th>
<th>Criteria</th>
</tr>
</thead>
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<tr>
<td>1.</td>
<td>Maximum Span Length</td>
<td>50.0 m</td>
</tr>
<tr>
<td>2.</td>
<td>Minimum Span Length</td>
<td>50.0 m</td>
</tr>
<tr>
<td>3.</td>
<td>Maximum Cross Slope</td>
<td>2.50%</td>
</tr>
<tr>
<td>4.</td>
<td>Maximum Weight of Segment</td>
<td>85.0 T</td>
</tr>
<tr>
<td>5.</td>
<td>Maximum Length of Segment</td>
<td>3.0 m</td>
</tr>
<tr>
<td>6.</td>
<td>Minimum Horizontal Radius</td>
<td>Straight</td>
</tr>
<tr>
<td>7.</td>
<td>Total Height of Segment</td>
<td>3.25 m</td>
</tr>
<tr>
<td>8.</td>
<td>Segments Glued or Dry Jointed</td>
<td>Epoxy Glue</td>
</tr>
<tr>
<td>9.</td>
<td>Segment Feeding Method</td>
<td>First 3 Spans Bottom Feeding, Remaining Spans Rear Feeding</td>
</tr>
</tbody>
</table>

LG was assembled over the LHS of A2 embankment and auto launched for the erection in the first span as shown in figure 11. For the first three spans the segments were fed from the bottom of LG for erection as the land was available. Modular trailer of 100 MT capacity were used to shift the precast segments from stacking yard to erection position. Trailer will park below lifting point of LG. In case of bottom feeding the segment will be lifted from trailer by lifting hoist of LG. The gantry with segment is moved to the specified location and the segment were hanged using macalloy bars from bottom of LG as shown in figure 13.

In case of rear feeding system as shown in figure 12 the position of the lifting point will be maximum 11.50 m from rear main support. The segment will be lifted from trailer by lifting hoist of LG. The gantry with segment is moved to the specified location then lower the segment below the LG, rotate the segment by 90 degree (in Plan) and hang the segment using macalloy bars from bottom of LG. After lifting all segments as per defined sequence dry matching is done. Once dry matching is found satisfactory gluing and temporary stressing are carried out. After temporary stressing, permanent stressing is done for the span. Once permanent stressing is completed the complete span is lowered on the bearing and LG is auto launched for the next span. This process of erection continues till the LG reaches the last span to be erected as shown in figure 14.
Figure 11: Erection using GSS and LG.

Figure 12: Segment feeding done from the rear side of LG.

Figure 13: The feeding is done from the bottom.
4.1 Time cycle achieved using LG

The time cycle achieved using the Launching Girder is 6.76 days per span which is having a learning curve for the first four spans. The first four spans have taken more time as there was research work going on for checking the deflection and torsion problems in the truss. The effective time cycle achieved excluding the learning curve is 3.7 days per span. The process has streamlined from the fifth span onwards and the time cycle has been reduced to 3.1 days per span which is exceptionally good in terms of accuracy and operation of the LG.

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>Span</th>
<th>Segment erection</th>
<th>Alignment, dry match gluing &amp; Temporary stressing</th>
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<th>Final Stressing</th>
<th>AUTO LAUNCHING</th>
<th>Total Days</th>
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<td>2.9</td>
<td>2</td>
<td>1.9</td>
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</tr>
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<td>2</td>
<td>P14 - P13</td>
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<td>2.9</td>
<td>2.1</td>
<td>1.9</td>
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<td>0.5</td>
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<td>0.25</td>
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<td>12</td>
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<td>15</td>
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<td>0.25</td>
<td>0</td>
<td>2.6</td>
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</table>

Total Time taken for erection of 15 spans Days 101.41
Minimum Time Cycle per span Days 3.10
Average Time Cycle per span excluding the learning curve Days 3.70

5. PROCESS OF COMPOSITE SLAB

The cranes were used to erect the structural steel members in the last five spans as shown in figure 15. The equipments played a major role in the erection of composite slab in the project. The slab was planned using the Roca Rolla Deck Sheet. This sheet acted as a sacrificial shutter and reduced the time frame by deleting the
activities like staging and bottom shuttering. The erection of steel structure was a challenge in the completion of the project. The advance planning helped us to use the opportunity to erect the steel beams before the onset of monsoon. Once the flood level has raised it will be difficult to erect the steel structure and will result in delay in the project completion. The composite girders were shifted using the modular trailer as shown in figure 16. The decision to use Roca Rolla deck sheet as shown in figure 18 helped in faster completion of the project. This has eliminated the role of staging and bottom shuttering which is quite impossible to do considering the present conditions. The completed view of the bridge deck as shown in figure 19 makes each one of us feel proud to be a part of this project.

Figure 15: Erection of composite girder using cranes

Figure 16: Shifting of composite girder by modular trailer.

Figure 17: Composite slab top view of Ganga bridge.
7. RECOMMENDATIONS
The general recommendation for using the various erection process in a single bridge construction are
a. Proper planning and monitoring is must in terms of arranging the resources.
b. The availability of ground support will decide the use of GSS for erection.
c. For spans, more than 15 in numbers the selection of LG as erection method will be beneficial. As this
   is the faster mode of erection with less difficulties as compared to other modes of erection.
d. Every erection process has a learning curve in the initial phase of construction based on the
   availability of resources at the point of use. This needs to be as small as possible for effective use of
   the equipments in erection.
e. Proactive thinking in the projects will help reduce the lag time in gaining the pace.
f. Parallel activities shall be identified and should be taken up immediately so that the work completes
   at a faster rate than expected with less defects.

7. CONCLUSION
Proactive planning is part of several distinct elements of business, including management, maintenance and
public relations. The difficulties in the erection process of box segments over Ganga bridge were anticipated
well in advance in the project and were taken up effectively. The change in methodology for erection of box
segments has helped the project to save time by deploying GSS and LG parallel in the project. The
availability of GSS in the project was helped in controlling the cost as with slight modification made in-house
it was made suitable for the project. It was observed that by the time GSS has completed 10 spans on the RHS
side LG also covered five spans in the LHS side. As the erection of LHS & RHS were independent of each
other the gain in time was almost 40%. Generally, the time cycle considered for calculation using GSS is 6
days per span and LG 3 days per span. But in this project, we are able to achieve minimum time cycle of 4.0
days per span using GSS and 3.1 days per span using LG which is remarkable achievement in the industry.
The project has been completed in such a fashion as if the erection was done using two LG. By completing this project by less than 24 months Afcons has demonstrated its capability in successfully delivering large value EPC projects.

8. ACKNOWLEDGEMENT

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Abstract:

Construction of bridges over railway lines is always a difficult task considering their importance and strict norms of the railway authority in India. In such scenario, constructing a steel bridge over one of the busiest railway lines, running the most important express-trains of the country, was a challenging task. The timely completion of this steel bridge was crucial to meet the deadlines of one of the most ambitious projects - The Agra-Lucknow expressway, recently completed in the country.

The proposed steel bridge consisted of four steel trusses, out of which two trusses were over busy railway lines. Each steel truss was 87.6m long, 20.65m wide, with 44 degree skew angle. It was impossible to erect the steel truss by conventional methods in the 75 minutes block time given by the railway authority. So it was decided to erect these steel trusses on adjacent spans and roll them on elevated supports over the railway lines to their final position. This paper gives the methodology adopted for launching of the steel trusses to reach its precise location in the imposed time constraint.
Fast track erection of the largest skew steel truss bridge of India

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1. INTRODUCTION

The Agra-Lucknow expressway is considered one of the most ambitious infrastructure projects in India. After its completion, the travel time between Agra and Lucknow has been cut down from six hours to three-and-a-half hours. Apart from strengthening the connectivity, the expressway has also bolstered economic growth in the Indian state of Uttar Pradesh. But the construction of this six lane expressway in the time schedule of two years was not an easy task. Afcons Infrastructure Limited was awarded the task of completion of 126 km out of the 302 km stretch of the expressway. This involved construction of a 750m long bridge over the river Ganga, 8 flyovers, 23 underpasses, 29 minor bridges and a rail over bridge (ROB).

Construction of bridges over railway lines is always a difficult task considering their importance and strict norms of the railway authority in India. In such scenario, constructing a steel bridge over one of the busiest railway lines, running the most important express-trains of the country, was a challenging task. The timely completion of these steel ROBs was crucial to meet the deadlines of the project.

The proposed steel ROBs consisted of four steel trusses, out of which two trusses were over busy railway lines. Each steel truss weighed 1500 tonnes and was 87.6m long, 20.65m wide, with 44 degree skew angle in plan (Refer figure 1).
Figure 2: Actual site photograph

The simplest method of erection would be by directly assembling the truss over railway lines by using a crane. But this was impossible in the 75 minutes block time given by the railway authority. So it was decided to erect these steel trusses on adjacent spans and roll them (launch) on elevated supports over the railway lines to their final position. The methodologies considered for erection of the trusses are given below.

2. METHODOLOGIES FOR LAUNCHING:

i) Methodology 1: The earlier methodology was to fix rollers on temporary supports and roll the truss over it. But this methodology was later discarded since there were high reactions on the bottom chord member of the truss. The bottom chord member would require to be heavily stiffened for these high concentrated loads.

ii) Methodology 2: In this methodology, rollers were mounted at each bottom node point of the truss. This evaded the problem of high concentrated reactions on the bottom chord member, in the earlier methodology. The top surfaces of the temporary supports were fabricated such as to offer smooth entry and exit of rollers over them. This scheme was finalized and got approved by the railway authorities for final erection.

3. COMPONENTS FOR LAUNCHING:
The launching operation requires many structural and electro-mechanical components. The major components for push launching are described below:

i) Push Launching supports:
To ensure the stability of the truss during the different stages of launching, temporary supports are required to be erected at intervals. These supports provide surface for the movement of rollers and so were called Rolling track supports (RTS, Refer Figure 3).

In the launching of the steel ROBs, two sets of six RTS were provided, one on each side of the truss, for launching operation of a single truss. These supports were also used for erection of the truss. A strip of high grade steel was welded on the top of RTS due to requirement of high bearing stresses from rollers. The top surface of RTS was profiled to allow smooth entry and exit of rollers over it (Refer Figure 4).
ii) Nosing truss:
The nosing truss is a truss connected at the forward/rear side of the truss to increase stability during launching. The nosing truss has lesser self weight as compared to the main truss. The nosing truss also helps in reducing the forces in the main members of the truss and also the loads on the rollers.

In this launching, a nosing truss of 15m length was connected at the forward launching end of the truss (Refer Figure 5).

iii) Rollers:
Since, rolling friction is less than sliding friction, we have preferred to use rollers for moving the truss. The coefficient of friction of the rollers was an important parameter for selection of the rollers. For this reason, Hilman rollers were used for their low coefficient of friction of less than 5 percent. Also, Hilman rollers are very compact in size and weight.

The maximum load at each node point was found out during various stages of launching and accordingly, Hilman rollers of appropriate capacity were attached at each node point (Refer Figure 6).
iv) **Wire rope:**
Wire Rope of diameter 32 mm and grade 1570 conforming to IS 2266:2002 was used for the pulling operation.

v) **Winches:**
Winches were used to provide the pulling force for launching. Since the speed of launching required was more due to only 75 minutes block time, two electric winches (Refer Figure 7), with line pulling speed of 3.5m/min and capacity 10 tonnes, were used.

vi) **Sheave Pulleys:**
To reduce the capacity of the winch, sheave pulley blocks were used. Total four numbers of 3-sheave pulleys (Refer Figure 8), two at the back of the truss and two in the front, were used for the launching operation.

The total weight of the truss was 1500 tonnes.

Due to 5% friction offered by the rollers, total pulling force required at one end was equal to,

\[
\frac{1}{2} \times 0.05 \times 1500 = 37.5 \text{ tonnes, say 60 tonnes.}
\]

Due to six falls of rope on three sheave pulleys, pulling load was reduced to \(\frac{60}{6} = 10\) tonnes. So, two electric winches of capacity 10 ton each were used on either side of the truss.
vii) **Anchor beam:**

Two anchor beams are connected on either side of the truss at the rear end, for attaching the two 3-sheave pulley blocks (Refer Figure 9).

![Figure 9: Anchor Beam](image)

viii) **Pier Brackets:**

Two brackets were connected to the central pier, on either side of the truss, for transferring the horizontal pulling force to the pier support. The two front 3-sheave pulley blocks were connected to the pier bracket (Refer Figure 10).

![Figure 10: Pier Bracket](image)

ix) **Guide Rollers:**

There is a possibility of the truss to move in the lateral direction during push launching due to reasons such as skewness of the truss, unequal pulling force at both ends of the truss, more friction on one side, etc. To avoid excessive lateral shifting, guide rollers mounted on brackets (Refer Figure 11), were provided on RTS. The guide brackets were designed for 10 percent of the maximum vertical load.
4. DETAIL METHODOLOGY

Stage 1 (Refer Figure 12)

i) All the RTS were first erected on pile foundations, followed by erection of assembly trestles.

ii) Speed restrictions were applied to trains during the erection of RTS5 support which was close to railway track.

Stage 2 (Refer Figure 13)

i) The main truss members alongwith nosing truss members were erected over timber packing placed on assembly trestles and RTS.

ii) After complete erection of main truss, rollers were connected at each bottom node point of the truss.

iii) Anchor beams were connected to the main truss and pier brackets were erected.

iv) The entire assembly was jacked up and all timber packings were removed. The truss was then lowered such that the truss would be resting on only RTS supports through rollers connected at nodes.

v) Wire rope was reeved on the pulleys and finally connected to the winch, as shown in figure 14.
Stage 3 (Refer Figure 15)

i) The truss was supposed to be launched in two phases. In first phase, truss was launched by 37.5m.

ii) During this launching no railway block was required since truss was crossing over DFCC tracks.

iii) The erection of the truss, behind the truss being launched, could be started after the completion of first phase of launching.

Stage 4 (Refer Figure 16)

i) In the second phase of launching, truss was launched by 50.1 m.

ii) Block had to be taken from railway during this phase of launching.

Stage 5 (Refer Figure 17)

i) After the truss reached its final position, nosing truss was dismantled.

ii) The rollers were removed from node points and misalignment if any due to lateral shift was corrected.

iii) The truss was then lowered on bearings.

iv) The erection of the back truss was also simultaneously completed.
Stage 6 (Refer Figure 18)

i) All the assembly trestles and RTS were dismantled, followed by casting of deck slab on deck sheet.

Deflection mitigation

i) The lateral deflection of the truss was restricted by the guide rollers on RTS.

ii) The vertical deflection of the truss was catered by attaching jacks mounted on rollers, at the tip of the nosing truss. In case the front rollers of the nosing truss went below the rolling track surface, the jacks on the nosing truss were operated till the front rollers come above the rolling track surface. The truss was then moved forward and the jack was again closed so that the entire load was transferred to the front rollers of the nosing truss (Refer Figure 19).
5. MAJOR CHALLENGES
   i) Precise survey was required throughout the project. Initially survey was required to locate position of all RTS. Also, after erection the top surface of all RTS were required to be at same level with tolerance of +/−5mm. Changes in the location and levels of RTS would have considerable effect on the loads of each roller. Also continuous monitoring of the deflections of the truss was required during the launching operation.
   ii) Strict tolerances had to be followed for fabrication of all RTS, as they were the major load taking members during launching and their final level after erection had to be same.
   iii) Assembly of the truss was another challenge due to the geometric complexity of the truss. Also after complete assembly of the truss, a camber of 100mm was required at the center of the truss.
   iv) For erection of RTS support, site had to work with caution due to its close proximity to the railway lines with high voltage overhead wires.

![Figure 20: Photograph of completed truss](image)

6. OBSERVATIONS/CONCLUSIONS
   i) The launching was successfully completed in the block time of 75 minutes given by the railway authority.
   ii) The speed of launching observed was 0.5 to 0.6m/min.
   iii) The average pulling force required during launching was 1.8 to 2.2 tonnes (coefficient of friction equal to 1.4% to 1.8%). The maximum pulling force was 6 tonnes, which was observed when the rollers were climbing the top girder of RTS during entry.
   iv) The guide rollers proved very important during the launching operation as the truss was being guided by the guide rollers on lateral shifting. The maximum lateral shift of the truss after reaching final position was observed to be 70mm.

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8. REFERENCES
   i) Hilman Roller Brochure for Engineered Products & Systems for heavy moving projects.
ABSTRACT:
On an urban freeway, the major stream of traffic traveling near the diverging area is frequently imposed to slow down or stop due to exiting traffic to the interchange. As the congestion concentration for queuing vehicles in the exit lane increases, the jam gets transferred to the main direction of travel resulting in a major hindrance for free flowing vehicles. In other words, the vehicles waiting to take the exit at the adjacent lanes right before the exit location blocks the vehicles traveling on the major lanes, causing a shockwave. A proper separation of two groups of traffic on the diverging segment of the urban freeway is necessary to maintain higher level of the service of the urban freeway and to improve traffic efficiency and safety on the diverging segment of the highway. In this study, on-lane traffic control device has been introduced to provide stronger separation of the traffic movement on the exit location. This technology embeds the LED lights on the lane between exiting lane and major freeway with green, yellow, and red color controlling allowance of the lane change near the exit location to minimize interference from the sudden cut-in on the queue of exiting traffic. The use of solar powered LED lights is expected to greatly enhance the coordination of moving vehicles when taking the highway exit detour.
On-Lane LED Traffic Control
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1 INTRODUCTION

Exits over the length of a freeway pose a threat to the stream of through-travelling vehicles traversing near areas where diverging occurs. As the number of queuing vehicles in the exit lane increases, the jam gets transferred to the main direction of travel resulting in a major hindrance for free-flowing vehicles. The large crowded number of vehicles waiting to take the exit will influence the vehicles travelling in the forward direction to overtake vehicles right before the exit location, which creates a shockwave in the forward main direction of travel. Therefore, innovative and sustainable methods that employ intelligent technology systems (ITS) are key in minimizing congestion and drivers’ travel time. These innovative ideas can be vital in improving the level of service (LOS) of the road by providing a safer and more efficient highway movement for inflowing and outflowing commuters.

The area of study chosen to simulate this shockwave effect is located at Exit 39 in Sheikh Zayed Road in Dubai, which is the exit leading to the Mall of the Emirates. This area is depicted in Figures 1 and 2. The goal is to limit the effects of congestion caused by this exit to the exit lane alone. This way, the queue will have a higher chance of clearing up without any external vehicle interruption or significant risk.

Figure 1. Sheikh Zayed road Exit 39.

Figure 2. Study area (Exit 39).
When exiting a highway, a right exit taper is regarded as a standard permitted diversion route for vehicles to take. These common operational techniques are the ones that are usually implemented, since road users find them more familiar. Increasing the number of lanes can increase the LOS of the system. However, due to limited space and economic constraints, increasing the number of lanes is not always a feasible solution to the congestion. Moreover, the delay time is regarded as the total time the vehicles spend in the system while waiting to receive service. A higher delay time leads to a greater number of vehicles having to queue before they can take the exit. This happens because of the shockwave effect, which is caused by the overtaking vehicles and the arriving vehicles.

The north bound through traffic in Sheikh Zayed road is affected due to the traffic congestion created at Umm Suqeim road exit (East bound) during the peak hour. The solution selected for this problem should be an innovative engineering design that enhances safety, mobility, and sustainability. The use of solar powered LED lights along with the non-mechanical lane markers is expected to improve the coordination of moving vehicles when taking the highway exit detour. These LED markers can be used to prevent vehicles from changing lanes close to the exit. This would improve the delay time and the safety of drivers because it limits the danger of overtaking vehicles in case of heavy congestion. The proposed On-lane LED traffic system influences the driving patterns observed in the road. The number of vehicles which would be inclined to change lanes near the exit ramp can be reduced. The implementation of this solution promises fewer congestions concentrated in the exit ramp which would improve the traffic in the through direction as well.

Road detectors can be used to know the volume of vehicles present and the volume of vehicles measurement is obtained by using road detectors, and when there is no congestion at the exit lane, the installed light panels will be turned off to save energy. When the exit lane starts to get congested, a flashing red light will appear on the road surface to indicate that in a brief period, changing lanes in this segment of the road will be prohibited. After the lane is fully congested, the LED lights will turn to permanent red lights to indicate that the lane's capacity is full and crossing to exit lane near the exit area is not permitted anymore.

2 METHODOLOGY

By using VISSIM to simulate the scenario and by assuming peak hour flow to commercial areas such as Mall of the Emirates, it can be noted that the flow in the through direction is heavily obstructed due to changing lanes to reach the right lane. This is because the vehicles going to the mall need to take Exit 39. This is shown in Figure 3.
After adding the LED lighting systems to the exit lane dashed markings, they serve as a warning by flashing red when the road is approaching its full capacity. They also act as a continuous lane marker when the lights flash continuously, which means they would resolve the congestion by making crossing prohibited. These LED lights will reduce the risk associated with congestion and are automatically activated through sensors that detect congestion. The resolved congestion is shown in Figure 6. A longer span for the LED layout shows greater control and even less congestion. This is shown in Figure 7.
It is important that those LED lights should be environmentally friendly and economically feasible. Solar panels can be used to supply those LED lights with electricity during the peak hour. A drawing of the red LED lights is depicted in Figure 8.

When the LED lights are operating, the LOS for the exit and through movements are not affected by each other anymore. Assuming the LOS for the exit lane is F since its fully congested at the peak hour, having the LED lights turned on to red will prevent any crossing from the adjacent lanes, and the effect on the through vehicles will be removed the moment the lights are completely red. Therefore, the through movement will get back to its natural flow and its LOS at the peak hour will stay the same as it is at any other time during the day.

3 State of the Art

To implement LED lights for this application, the installation and operational cost and feasibility must be studied beforehand. These costs must be weighed against the benefits of employing this technology. The economic benefits of using LED lights in this field are apparent in different transportation applications. The U.S and various European countries have been replacing traffic lights with LEDs since the 1990s. This is
mainly because of the significant cost savings due to this replacement. A project, which was launched in Sao Paolo, replaced common traffic fixtures with the high-power LEDs shown in Figure 9. The project has reduced signal related energy usage in Guarulhos by nearly 90%. 1340 megawatt hours are saved per year as a result of this project. In fact, the initial cost was approximately USD $750,000 (R$1.35 million) and the benefits exceeded this cost only after 12 months. (Philips 2008) By taking advantage of these present LED technologies, energy consumption can be prioritized and the implementation of the proposed solution would be more feasible.

![Figure 9. LEDs for application.](image)

To detect congestion, pneumatic road tubes are proposed. Figure 10 shows the use of rubber tubes placed across the street lanes for vehicle data collection, most importantly the number of vehicles and the speed of vehicles. For one-way road lanes, the speed data is collected by having a set distance between two tubes, and the time is noted when one-axle crosses the first tube by noting the change in the tube's air pressure. The tubes are connected to the same counter. When the same axle crosses the second tube, the initial cross and final cross timings are used to find the time difference. This is to determine the speed of the vehicle, which can be found by using the known distance between the tubes and the collected time. For two-way roads, the direction of the vehicle is noted by the first tube crossed. In addition, raw time data is converted into time intervals, vehicle classification, and speed. According to FHWA classification of vehicles, vehicle classification varies depending on the number of axles, the spacing between axles, and the number of units attached. On the other hand, those tubes have some disadvantages. For instance, if two 2-axle vehicles cross the tube quickly with small space between them, the tubes might count the 2 vehicles as a 4-axle single unit tractor with trailer. Pneumatic tubes can be used in this project to determine the number of vehicles in the exit taper. These tubes will determine the change in the volume of the lane at the peak hour. (McGowen & Sanderson 2011)

![Figure 10. Pneumatic road tubes.](image)

This system of LEDs can be improved in the future as it is compatible with a lot of future innovations such as LiFi (Light Fidelity technology). Several methods such as traffic signal controls, traffic
Routing, turning restriction, and congestion pricing may be applied to alleviate congestion in roads during peak hours. Traffic routing is basically directing the traffic flow towards using the less congested routes. It can be implemented by using a LiFi. A LiFi can be used by installing transmitters on traffic signals and on the rear part of vehicles. Also, a possessive receiver at the front part of each vehicle should be installed. Different light colors indicate different data transmitted to the receiver which allows the transmitter at the rear of the vehicle to transmit data to other receivers on other vehicles. For instance, a break will alert all receivers to slow down. Figure 11 shows the display system inside the vehicle. Furthermore, using the Internet of Things component helps in rerouting and predicting traffic congestion. This component connects sensors on road, traffic cameras, and Bluetooth data from mobile phones and stores the collected data in a local cloud that predicts areas of congestion. It also shares the data with all potential users to avoid heading to the congested areas at the peak hours. (Krishnan & Balasubramanian 2016) These components can be applied along with the proposed LED technology in cases like the Sheikh Zayed road case study. Because LiFi uses LED lightbulbs for data transmission, the LED markers proposed can be integrated as a part of this system.

Figure 11. Display screen inside the vehicle. (Krishnan & Balasubramanian 2016)

Static Early merging to the open lane allows a driver to merge in advance before reaching the end of the queue, and that reduces merge-related and rear-end collisions. The frequency of forced merging near the exit is also reduced. Also, dynamic early merging is a useful technique in situations such as an exit taper from freeways or closed lane merging to an open lane. When the vehicle is detected stopping on the open lane, a signal is transmitted to send flashing signs to the vehicles on the closed lane not to pass. When the vehicles are moving consistently on the open lane again, the flashing signs turn off. Static and dynamic early merges are shown in Figure 12.

Figure 12. Static merge. (Pesti, Wiles, Cheu, Songchitruksa, Shelton, & Cooner 2008)

Moreover, late merging is an effective merging control that allows vehicles to stay in the closing lane up until reaching the merging point as shown in Figure 13. It is better than the early merge since it reduces queue length by 50%, and decreases the rage among road users by allowing them to select their desired lane (the most empty lane) in order to reach the merging point. Studies show that the late merge is better than early merge during peak hours, but at low volumes, early merge is better. Therefore, dynamic late merge is developed by switching between early and late merges. When the detectors detect high volume on the open lane, dynamic merge control works by switching electrical sign boards on to keep vehicles in the open lane, and another sign will encourage lane change at the merging point. When the volume is low, the sign will either switch off or give advising message to stick to the speed limit besides to the static sign boards that enables static merge. In fact, the application of early, late, and dynamic late merge control in this project is highly possible, but it will be using LED flashing light systems. Dynamic late merging control is to be applied during peak hour since the flashing and permanent red LED lights are activated when the exit taper
starts to get congested and gets fully congested respectively. (Pesti, Wiles, Cheu, Songchitruksa, Shelton, & Cooner. 2008)

Alleviating traffic congestion greatly contributes to the reduction of CO₂ emissions by traffic. This is because the congestion causes the cars to follow a stop-and-go pattern, which increases the emissions. By using technologies such as the LED technology suggested in this paper to reduce, congestion, the number of acceleration and deceleration events is reduced. This is shown in various studies conducted. Studies show that reduction in congestion reduces emissions, fuel consumption, and risk of premature mortality. “Real-World CO₂ Impacts of Traffic Congestion” by Matthew Barth and Kanok Boriboonsomsin discusses the impact of congestion on CO₂ emissions. To study vehicle emissions, the authors developed a microscale model, Comprehensive Modal Emission Model (CMEM), which can predict second-by-second vehicle fuel consumption and emissions based on different traffic operations. Vehicle trajectories can be applied to the model to produce individual and aggregate emissions estimates.

One way to estimate the impacts of congestion on emissions is to examine the velocities of vehicles under different levels of congestion. On a freeway for example, vehicles typically have a high speed that does not vary a lot. However, as the density of vehicles increases, average velocity tends to decrease and individual vehicle velocities start to fluctuate. Figure 14 shows vehicle velocity patterns for different congestion LOS on a freeway. As LOS conditions get worse, vehicles travel at lower average speeds with more fluctuation. For Figure 14, it is possible to estimate fuel consumption as well as CO₂ and pollutant emissions using the CMEM model. (Barth & Boriboonsomsin 2008)

A database of vehicle activity on freeway was applied to the CMEM model. This set of vehicle activity data was collected by probe vehicles on freeway mainlines in Southern California. In Figure 15, the estimated CO₂ emissions are plotted as a function of average running speed. Figure 15 also illustrates CO₂
emissions for steady-state speeds, which is considered as the lower bound of CO₂ emissions. In reality, all cars experience stop-and-go, which causes higher emissions.

![Figure 15. CO₂ emissions (g/mi) vs. average trip speed (mph). (Barth and Boriboonsomsin, 2008)](image)

The results show that whenever congestion in a freeway brings the average vehicle speed below 72 km/h (45 mph) there is a negative net impact on CO₂ emissions because vehicles spend more time on the road. However, if the congestion is not severe and it brings down the average speeds down from the free-flow speed of 105 km/h (65 mph) to a speed of 72 to 80 km/h (45 to 50 mph), this moderate congestion can actually lower CO₂ emissions because extremely high speeds beyond 65 mph can increase CO₂ emissions. This shows that under a speed limit, strategies to reduce congestion will most certainly reduce CO₂ emissions. To limit the excessive speeds, speed management techniques can be used. To limit the congestion, congestion mitigation strategies and traffic flow smoothing techniques can be used. (Barth & Boriboonsomsin 2008)

Another problem caused by congestion is that it increases fuel consumption. In "Measuring the Effects of Traffic Congestion on Fuel Consumption" by Alvaro Garcia-Castro and Andres Monzon, an analysis is performed to relate traffic congestion and fuel consumption. This uses delay rate as the indicator of congestion. The results for urban areas are shown in Figure 16.

![Figure 16. Relation between fuel consumption and delay rate. (Garcia-Castro & Monzon 2014)](image)
Figure 16 shows Positive Accumulated Acceleration (PPA) and fuel consumption as a function of delay, for an itinerary of M30 urban motorway. PAA is an indicator defined by the authors, Garcia-Castro and Monzon. It represents the area under the positive accelerations curve, so a trip in which the speed fluctuates a lot tends to have a relatively higher PAA value than a trip with the same route (itinerary) but constant speed. The data shows that a small increase in traffic from free flow conditions is good in terms of fuel economy. However, the general trend remains that congestion increases fuel consumption. So, there is optimum fuel consumption as a function of the delay rate. Once the optimum is exceeded, increasing traffic intensities cause speed profiles to vary and the fuel consumption increases. (Garcia-Castro & Monzon 2014)

Furthermore, the impacts of congestion on public health must also be considered. “Evaluation of the Public Health Impacts of Traffic Congestion: A Health Risk Assessment” by Jonathan I Levy, Jonathan J Buonocore, and Katherine von Stackelberg is a study of the impact of congestion associated on the concentration fine particulate matter (PM2.5) in the air. The article also evaluates the public health risks associated with these particles. The study was performed across 83 urban areas. The results show that the VMT is projected to increase 33% from 2000 to 2030. Trajectories of the degree of congestion by urban area closely pattern trajectories of VMT, although not in a linear fashion. In 2005, traffic emissions attributable to time spent in congestion include approximately 1.2 million tons of NOx, 34,000 tons of SO2, and 23,000 tons of PM2.5. These emissions are associated with approximately 3,000 premature deaths in 2005. This is shown in Figure 17.

Figure 17. PM2.5-related premature mortality attributed to traffic congestion in the USA. (Levy, Buonocore, & Von Stackelberg 2010)

The decline in mortalities shown in Figure 17 is due to reductions in emissions per vehicle. However, after 2020, the projected decline ends and there is a steady increase in premature deaths. This is due to the fact that the per-vehicle emission reduction is expected to reach a plateau while the population will continue to grow. (Levy, Buonocore & Von Stackelber 2010)

3 CONCLUSIONS

This paper provides a suggested solution to the effects caused by a congested exit lane on the through traffic of a highway. LED lights can be used with the lane markers to limit the time where vehicles can change lane close to the exit. When the demand becomes equal to the capacity, the lights will turn on, which prohibits vehicles from changing lanes. Vissim simulation concludes that this method is effective in reducing the congestion and the number of accidents, pneumatic tubes would be used on the road to collect data and to sense congestion. Positive effects of this solution are all effects of the reduced congestion on the environment. Literature review shows the importance of reducing congestion and delay time on saving fuel and reducing CO2 emissions. A reduced congestion also has a positive effect on public health.
REFERENCES


Road Data as Prior Knowledge for Highly Automated Driving

In highly automated driving processes, the vehicle must recognize and record the roadway with the help of existing sensors (stereo cameras, radar, lidar, laser scanners etc.) and the associated driving area elements and convert them into a digital 3D-model in real time. The vehicle can then locate and orient itself and move in the so-called obstacle-free and restricted 3D-area. Localizing the vehicle precisely in the surroundings is often difficult because of the volume of data needing to be processed in real time, the accuracy of the object recognition process and the multiple disturbances like the weather, daytime and nighttime, the traffic situation etc. To gradually solve the object recognition problems in real time when depending on the available sensors and disturbances, the vehicle should have a detailed prior knowledge of the traffic infrastructure on the planned route through highly developed maps within its navigation system before the journey starts. By comparing the prior knowledge and the knowledge obtained from the surroundings, the localization of the vehicle can take place faster and more accurately. When selecting a route, the prior knowledge about the existing roads would be directly retrievable and could accelerate the localization process in the surroundings considerably and particularly make things safer if any problems occur. The digital data on roads and surroundings can also be used to calculate recommended speeds, the necessary distances between vehicles and other details for fully automated driving.
Road Data as Prior Knowledge for Highly Automated Driving

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1 THE PROBLEM

In any highly automated driving process, the vehicle needs to recognize and record the crucial driving area elements with the help of the existing sensors (stereo cameras, radar, lidar, laser scanners etc.) and convert them into a digital 3D model in real time. The located vehicle can then find its way and move on its own within the so-called obstacle-free restricted 3D area. It is often difficult to precisely localize the vehicle in the surroundings because of the volume of data needing to be processed in real time, the accuracy of the object recognition and multiple disturbances like the weather, day and night or the traffic situation etc. In order to be able to gradually solve the object recognition problems when depending on the available sensor technology and the disturbances, the vehicle should have a detailed prior knowledge of the traffic infrastructure on the planned route with the help of extended digital maps in the navigation system. By comparing the prior knowledge and the knowledge gathered, it is possible to localize the vehicle faster and more accurately in the surroundings.

In addition to precisely localizing the vehicle, kinematic and vehicle handling parameters derived from the road geometry can be made available to the automated vehicle for the automated driving process. Selected reference variables here are e.g. the recommended speed for a bend or the necessary distance for any emergency braking maneuvers that are required.

By specifying the speed profile for the section of road where the vehicle is traveling, once a route has been selected, the automated vehicle can handle the longitudinal control process itself, depending on the geometrical parameters on the horizontal projection. The recommended speed for driving along a bend is largely dependent on the equilibrium of forces in the vehicle’s handling. On straights, on the other hand, the speed is normally determined by the vehicle’s engine power and the permissible maximum speed in line with the current road traffic regulations.

2 HIGHLY AUTOMATED DRIVING

2.1 SUMMARY

The longitudinal and lateral control of the vehicle (acceleration or braking or overtaking or making a turn) is increasingly transferred from the driver to the vehicle control systems. Depending on the degree of automation, it is possible to classify five fundamental development stages for automated driving (Fig. 1). While driving a vehicle without the intervention of vehicle control systems (normal driving) is increasingly becoming a thing of the past, assistance systems are increasingly helping it on the three levels of stabilization
(ABS, ESP), driving (ACC, LCA) and navigation (routing systems). By introducing new kinds of sensor technologies, the vehicle can take over the longitudinal and cross slopes (gradient and camber) for a certain time and the driver then monitors the driving process. However, the aim is to guarantee fully automated driving through using suitable vehicle control systems with a high degree of reliability where the driver only has to take over control in exceptional situations or not at all.

2.2 3D MODEL

In order to be able to solve the object recognition problems depending on the available sensor technologies and the disturbances (weather, vision), the vehicle should have a detailed prior knowledge of the traffic infrastructure on the planned route with the help of extended digital maps in the navigation system. By using a real-time comparison between prior knowledge and gathered knowledge, it is possible to provide faster and more precise localization in the 3D environment.

The special requirements for road data from the point of view of highly automated driving have been analyzed within a research project [1] and, at the same time, a fundamental methodology for logging, processing and preparing road data for further developed has been suggested. By introducing special features for different degrees of detail on the roadway and the associated elements that characterize the driving area, a standard data structure with the associated data formats is possible, which can then be transferred to digital road maps.

![Figure 1. Development stage of automated driving](image)

If the responsible road planning authorities keep to the methodology needing to be developed in their planning, construction and final surveys, the map producers would be able to directly transfer all the necessary 3D data from the complete process into their navigation systems. When selecting a route, prior knowledge from the available road data could be directly retrievable and could therefore significantly accelerate the localization process in the surrounding area and particularly make things more reliable if any disturbances occur.
3 ROAD DATA

3.1 GENERAL MATTERS

3D models on the roadway normally already exist for all the new building, conversion and upgrading work, which has been planned and completed in Germany since 1990, as a result of the design planning. Unfortunately, the 3D models have not normally been updated as part of preparing the “as-built” plans once the building work has been completed – i.e. the existing plans resulting from surveys of the current state do not always include the updated 3D roadway models. As the surveys of the current state do not always follow a standard surveying catalog, no digital transverse profiles are available for a direct data transfer process in the end either.

Only horizontal projections with individual height coordinates with and without special information on the design parameters on the horizontal projection normally exist for older roads.

3.2 DEGREES OF DETAIL

In order to be able to standardize the road data, it is necessary to introduce roadway models with different degrees of detail depending on the special application. The following different degrees of detail have been derived from the research project (Fig. 2).

Depending on the case at hand (driving simulation or real automated driving), the road data must be produced in a suitable degree of detail for the road section, i.e. in addition to using 3D models from the design planning work and the 3D processing of existing plans, the missing data information for individual routes needs to be recorded using the measuring vehicles too.

![Figure 2. Standardized degrees of detail for the roadway with the surrounding features too](image-url)
3.3 DATA LOGGING AND PROCESSING

The data logging and processing takes place in the following 3 separate stages:

- Inspecting or assessing the existing design plans according to the existing 3D geometric models with different degrees of detail
- Supplementing 3D geometric models by re-routing on existing plans
- Recording and calculating the 3D geometric models with associated driving area information using a measuring vehicle.

An overall 3D model is developed from this data by superimposing it with the suitable degree of detail.

3.4 MEASURING VEHICLE FOR DATA RECORDING

A new kind of measuring and testing vehicle is currently being developed as part of a different research project [2]. The basic vehicle is a Porsche Panamera (Fig. 3). An identical vehicle is already available at the Institute for Energy and Transport’s driving simulator laboratory and it is being used to compare experiments between real and virtual journeys.

In addition to recording road cross section data, the vehicle can also be used for real journeys to objectively assess driving behavior. Comparative experiments on driving behavior have already been completed for planning and constructing the in-field route at Porsche AG Leipzig [3].

Figure 3. Components in the measuring and testing vehicle and digital recording of road data using line lasers, ultrasound sensors and stereo cameras
4 PRIOR KNOWLEDGE

4.1 SUMMARY

3D models (transverse profiles) are calculated for the roadway as part of the design process – i.e. three-dimensional data for a road is available for approx. every 10 m and the building work takes place in line with this. The lane markings, the horizontal and vertical guidance devices (reflector posts, crash barriers and traffic signs) are available with the markings and road sign plan and can be linked to the data for the roadway to create a uniform 3D model.

When selecting a route for a journey with the navigation system, the already existing digital data about the section of the route could be retrieved as so-called prior knowledge by the automated management system and made available for the localization process. A constant comparison of the digital data from the acquisition of knowledge online with the stored prior knowledge then takes place in real time during the automated driving process (Fig. 4).

Figure 4. Comparing real-time knowledge and prior knowledge

In addition to the digital road data, a speed profile is available for the automated vehicle for the longitudinal control process and it already takes into consideration the kinematic and vehicle handling features and traffic law regulations along the route.

4.2. CRITICAL VELOCITY AND RECOMMENDED SPEED

The alignment of an existing, long-standing rural road on the horizontal projection is normally characterized by a sequence of non-coordinated circular arc and straight sections, which lead to driving behavior with a non-homogeneous velocity curve.

When proceeding from a straight section to a circular arc, centrifugal forces occur in the bend, which significantly affect driving stability and, in the end, can lead to the vehicle slipping. According to the “RAL” design regulations in Germany (Richtlinie für die Anlage von Landstraßen – Guidelines for Designing Rural Roads), one third of the compensation for the centrifugal force \( F \) takes place through the cross slope (camber) \( q \) of the road and two thirds by the transverse friction between the tires and the road surface (radial adhesion coefficient \( f_R \)).

Based on the vehicle handling interrelationships when travelling through a bend (Fig. 5), a so-called critical velocity \( v_{G} \) can be derived from the equilibrium of forces – and if this is exceeded, the vehicle will certainly start to “slip.”
The following applies to the equilibrium of forces when traveling through a bend:

\[ F \cdot \cos \alpha = f_R (G \cdot \cos \alpha + F \cdot \sin \alpha) + G \cdot \sin \alpha \]  
(1)

If the angle \( \alpha \) is small:

\[ \cos \alpha \approx 1 \quad \text{and} \quad \sin \alpha \approx \tan \alpha = q \]  
(2)

it is also necessary to use the following formula for the centrifugal force:

\[ F = \frac{G}{g} \cdot \frac{v_G^2}{R} \]  
(3)

As a result, formula (1) can be converted as follows:

\[ \frac{G}{g} \cdot \frac{v_G^2}{R} - G \cdot q = f_R \left( G + \frac{G}{g} \cdot \frac{v_G^2}{R} \cdot q \right) \]  
(4)

If we resolve the equation according to \( v \), the following type of formula is created for the critical velocity:

\[ v_G = \sqrt{\frac{g \cdot R \cdot (f_R + q)}{1 - f_R \cdot q}} \cdot \frac{m}{s} \]  
(5)

where

- \( v_G \): critical velocity for traveling through a bend, m/s
- \( g \): gravity acceleration, m/s\(^2\)
- \( R \): bend radius, m
- \( q \): camper, -
- \( f_R \): radial adhesion coefficient, -

Depending on the design parameters for the bend, the road surface features and the weather, the relevant critical velocity can be calculated for the bend. As a result, the speed profile can be transferred to the highly automated vehicle as prior knowledge for each rural road before the journey starts. It is necessary to ensure that a permissible maximum speed of 100 km/h should generally be followed on straight stretches on rural roads in Germany and any discrepancies can also be regulated by speed restrictions using traffic signs.

Fig. 6 shows the speed profile for the critical velocity and recommended speed depending on the individual elements on the horizontal projection (straights, bends) and the road surface (dry or wet asphalt) for the section of an existing rural road with a non-homogenous alignment on the horizontal and vertical projections, in addition to the gradient and the curvature features.
The critical velocity profile has been adapted for the purpose of harmonizing the driving behavior in the transitional area between a straight and bend, i.e. a recommended speed profile is therefore created for the automated longitudinal control process.

Figure 6. Critical velocity and recommended speed graph (speed limit on rural highways in Germany \( v = 100 \text{ km/h} \))

5 RESULTS AND OUTLOOK

The experiments conducted so far have clearly indicated that a prior knowledge of planned routes makes sense to reliably localize vehicles in highly automated driving scenarios. A uniform data structure, a suitable degree of detail and a transfer format for further processing in the vehicle are all necessary for this. The road planning authorities urgently need to start talking to automobile manufacturers in order to adapt the data structure in the planning process for road traffic facilities to the requirements of the automobile industry. Suitable control parameters for the automated longitudinal control process can be derived from the digital prior knowledge and they can be gained from the interrelationships between kinematic and vehicle handling dynamics and the weather conditions. Setting a recommended speed can significantly improve traffic safety on all rural roads with a non-homogenous alignment. Intelligent, digital traffic infrastructure can therefore make highly automated driving scenarios with suitable prior knowledge more appropriate for daily use and safer.
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Autonomous Drones in condition assessments and inspections of transportation infrastructure assets

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ABSTRACT

Aim/Objective:
To determine the efficiency of autonomous UAVs in undertaking assessments of the condition and performance levels of fixed and mobile assets under the jurisdiction of Transportation and Infrastructure Authorities.

Method:
- Establish prerequisite set of properties for drone autonomy including: (a) high accuracy GPS/GNSS and indoor positioning systems, (b) anti-collision and multi-sensor crash prevention system for all directions, (c) built-in “return to safe landing point” function through safe flight path, (d) real-time route optimization (battery life, remaining tasks, obstacle identification), (e) Real-time error handling, (f) Ability to fly as a team with other drones in formation.

- Elaborate on the optimal requirements for the transportation infrastructure asset inspections including: (i) The usefulness of drone survey generated 3D model(s) for the inspection area and all fixed obstacles (with proximity warning distances), (ii) The utilization of a well-trained object recognition model to recognize non-fixed obstacles (with proximity warning distances), (iii) The UAV equipment including high definition camera(s) (plus Lidar, thermal, infrared as required) and powerful on board GPU for real time object recognition (using pre-trained models with deep learning), (v) the application of a well-trained “condition assessment cases” recognition model.

Conclusion:
Review methodology against enabling technology currently available and suggest timeline for commercialization and scaling of such services
Introduction

The introduction of UAVs (Unmanned Airborne Vehicles), alternatively referred to as RPAS (Remotely Piloted Aircraft Systems) or most commonly known as Drones, is creating significant disruption in the Transportation Industry. Drones are gaining momentum continuously as Public and Private Sector organizations research, design and put to the test business applications using this technology which in many instances, is gradually becoming an integral part of their operations and services.

Drones were introduced commercially not long ago and were perceived as “games for adults” with a passion for aviation, robotics and recreational toy racing. However, the tremendous evolution of Information Technology and research undertaken in these fields by Academia, soon allowed access to Drones for a much wider audience. The media picked up the subject too and suddenly communities of Drone enthusiasts started flourishing all around the world. While the interest was growing exponentially, the Private Sector started seeing the potential of utilizing drones to do things faster, cheaper and from a distance. Naturally, the number of Commercial Drone manufacturing companies started to grow too and soon there were drones in the market with cameras, scanners, spraying nozzles, sensor readers and many more. In parallel what was in the past a “toy” had become a multi thousand-dollar device manufactured and marketed by companies who began appearing in lists with the fastest growing and most entrepreneurial Technology companies in the world.

All this happened over the past ten years and by this we do not refer to Drones; We refer to Drones becoming mainstream technology and the average person recognizing what a drone is without being surprised if he or she, sees one flying in the sky. It is of course known that Drones have been used widely and for a few decades now within the military and security services but in this paper, we will not touch this subject; we will focus purely on Transportation.

The big question is of course what will happen with Drones in the next ten years. There are tremendous expectations by organizations who have already adopted Drone technology. Aspirations such as “unlimited” battery and flight time, multi-weather and multi-environment (sea-air-ground) operation capability and significantly increased payload capacity (such as flying Taxis for people) to name a few.

The same time the list of challenges is growing as more Authorities commence UAV regulation and licensing programmes, initiate Policies and Laws and restrict dramatically the Fly Zones. Unfortunately, the initial introduction of Drones to the commercial and mainstream market, had very little regulation and there have been multiple incidents and accidents involving Drones around the world that led to many Authorities almost banning them. Additionally, issues around violation of privacy and unauthorized surveillance have risen dramatically to the extent that Hobbyist Drone pilots are often perceived as intruders and in many countries as criminals.

The positive note for the Industry, is that Regulation has made the subject very public through the media and all commercial drone services companies are now observing strict safety and operations rules while ensuring their staff is licensed, registered and properly trained. In parallel, within a growing number of Countries around the world, Drone enthusiasts and hobbyists are also obliged to register and get licensed. Some Authorities are going to further extents by establishing Drone Traffic Control systems and programmes that track the position of all Drones flying within their jurisdiction thus progressing to active enforcement if Drone Pilots do not observe and comply with the rules and regulations.

It is safe to say that evolution of Drone Technology and Drone Regulation will continue rapidly and that itself will be a big disruption for the Society, in general. However, when we consider specific Professional Services Areas like Transportation, it is the evolution of other parallel technologies and Sciences, building on existing technical specifications and established practices that when combined with Drone technology, cannot only disrupt but really start a revolution on how we will be “doing things in the future”, especially when it comes to asset inspections.

One of the most talked about subjects in Technology these days, is the introduction of Artificial Intelligence and Cognitive Computing into our lives. It starts with personalized digital assistants and extends to real time prediction modeling, on demand and targeted data/services, self-driving vehicles, self-thinking smartphones, self-goods ordering refrigerators, self-power adjusting street lights and so on! Naturally the Drone industry
could not be left behind. There is significant amount of software and hardware available for programmable flight paths and tasks, that is gradually becoming more and more available to a wide audience. The exposure of such capabilities of Drones, to a wider audience, is what is motivating the Transportation Industry to revisit and re-think existing processes, procedures and training curriculum around Inspections and Asset/Facility Management.

In the following sections, we will target an area that has significant technical and commercial context and that is the inspections of transportation assets and how we could potentially utilize Drones, equipped with advanced A.I. features, to make this task a lot safer, cheaper and faster. We will look into the context that allows a Drone to be called “autonomous” along with the aspects of health & safety features and constraints. We will then move on to what kind of inspection tasks can be performed using autonomous drones and conclude with the aspirations to realize all this as a mainstream commercial service that many of us believe, will be the standard way to undertake Transportation Asset Inspections in the future.

Drone Autonomy

Mobility and autonomy are fascinating subjects, especially when combined! The current technology advancement in these areas, especially within the automotive industry, is constantly a favorite subject amongst the Transportation Industry professionals and service providers but also amongst the ordinary people who feel they are getting closer to being driven to work by fully autonomous vehicles.

The Aviation Industry is following closely and autonomous flight operations are constantly gaining ground, especially when it comes to business and commercial applications for Drones. We will examine what this means for Transportation Asset Inspections.

The term autonomous “suggests” that a commercial drone used for condition assessments and inspections, can act independently or has the freedom to do so. Many commercial drones nowadays come with autonomous flight modes as a built-in feature. Practically for all Multirotor, Fixed Wing and Hybrid Drones in autonomous mode, the flight path and the mission are pre-programmed on the ground and the drone is using its GPS system to navigate across all mission waypoints and record video or images at certain timestamps and angles, as pre-defined by the mission programmer.

However, within the context of condition assessments and asset inspections, the term autonomous has a wider meaning and covers not only how the drone flies on auto pilot but also how it perceives and interacts with the surrounding environment, how it understands and constantly adapts to the mission objectives and how it knows what to do if things go wrong or the mission objective suddenly changes.

Therefore, in order for drones, used in condition assessments and inspections, to be characterized as “autonomous”, they should meet most, if not all of, the following four fundamental requirements that reflect what an autonomous human brain would do if it was “installed” on board:

- An autonomous drone should know where it is, at all times. An autonomous drone would fly most of the time in GPS stabilized mode and would use GPS to position itself and the gyroscope and accelerometer to maintain altitude and attitude. However, whether flying indoors or outdoors, there are challenges like GPS signal loss, if for example it is flying too close to a building. The vast majority of drones utilized in condition assessments and inspections, carry on board one or more high definition cameras that are used mostly for video recording and image capturing. Additionally, they have built in sensors that are used to avoid collision and bypass obstacles. There is growing technology nowadays and significant research in the field of video algorithms and analytics that combined with the ability to have on board a drone, a powerful GPU (Graphic Processing Unit), presents the industry with a unique opportunity in autonomous drone orientation and navigation.

This combination of different technologies can allow drones to utilize live sensor data, live video feed and optionally a set of preloaded spatial stereo models, to generate in real time, a 3D model of their surrounding environment. This model allows the drone to know with great accuracy and detail, which routes it can fly through, how fast it can go and what to avoid at all times. In other words, drones in the very near future will have, like humans do, their own “brains” and “eyes” to
comprehend their surroundings and always choose the safest and optimal routes to travel across the mission waypoints.

- An autonomous drone, after given a clear business objective and target for each mission, is able to identify the best route to follow and the optimal sequence of actions to complete all tasks it is given. It needs to continuously be able to adapt to and overcome any natural or fabricated challenges presented during a commercial flight. It needs to be able to continuously optimize the operation it is performing against remaining battery time, weather conditions, updated instructions during the flight and pre-defined performance KPIs.

- It goes without saying that any autonomous drone deployed for inspections and condition assessments, needs to have built-in sensor technology across all six directions, that allows for obstacles detection and avoidance, anti-collision and navigation away from environmental hazards and challenging weather conditions.

  It also needs to have built-in and pre-programmed safety procedures for all standard routine operations that take place during each flight (e.g. landing, take off, hover etc.) but also for a wide range of unexpected scenarios (e.g. loss of connection between the Drone and the Ground station/controller, signal interference, battery issues etc.)

- An autonomous drone should be able to fly as part of a group of Drones operating under a common mission. This concept of connected drones allows for large scale inspections where the assets are hundreds or thousands and communication/coordination between all drones in the same team, is the only way to complete the task with speed and efficiency

Summarizing, what would make a drone used for inspections, truly autonomous is the self-orientation at all times, the cognitive capability and interpretation of assets identity and condition, the adherence to strict safety standards combined with built-in technology to avoid obstacles/accidents and finally the ability to communicate and work together as part of a fleet serving a common purpose. That is in other words what a human would do if humans could ride an aircraft and fly through corridors of assets undertaking inspections!

**Not just a flying camera**

The initial excitement with drones was based purely on the fact that anyone could fly in places extremely inaccessible and record video footage and images for either recreational or commercial purposes. The concept of a flying camera that can go literally anywhere is up until now the “star feature” of drones but as multiple industries are bringing drones into their business domains, there is great need to add extra features and functionalities, that are targeted and specific to each business domain’s commercial requirements.

We will go through an overview of what drones can do nowadays, but also in the very short future, in the Transportation condition assessments and Asset Inspections domain.

- **Raw video footage and images.** The main parameters taken into consideration when scheduling asset inspections are urgency (proactive/reactive), health & safety, cost, resources and asset accessibility. There are of course a lot more that derive from modeling and strategy, but these five are always considered by asset condition assessors and asset management Engineers.

  When humans are assessing the condition of assets, the first sense they are using is sight. Once the inspector visits the asset, the first examination is always the visual one. In many cases this in enough to understand whether there are issues with the asset or not. In other cases, the inspector would need and use specialist equipment before coming to a conclusion.
There are some fundamental challenges on the ground when undertaking inspections. The first one is the accessibility and subsequently health & safety. No one is comfortable with having a crew climbing gantry signs and street lighting poles to assess their condition. Ideally humans should undertake such actions only if they are fixing, installing or changing fixtures on these assets. This is where drones have (or attempting to) solve the problem. A drone can produce high definition video and images of assets while flying around them. The ground drone operators can observe from a distance and assess remotely the condition of the asset being inspected. This of course an extremely cheap exercise to undertake. It is very fast and no human life is put in danger. Furthermore, it is an exercise that can be repeated multiple times and in most of the times, there is no effect on transportation and traffic operations since no scaffolding or special elevating equipment has to be brought in the middle of a road corridor.

The video and image data brought in, can be post processed and corrected using known control points and distortion removal techniques. The content can also be enhanced using specialized image and video processing software. Ultimately very detailed computer generated 3D models of the assets can be produced and provide the accessor with incredibly realistic details in a raster or vector editing environment. The images can be brought together, using geospatial software, to produce mosaics of whole areas and road corridors that allow for quick comparison of the situation between two or more inspection events. A few years back all this was just not practical since it could only be done using aerial imagery from planes and helicopters that was nowhere near as good as the imagery we get from drones nowadays. Additionally, the cost was so high that was prohibiting to even consider aerial imagery acquisition for condition assessments and inspections. As the technology becomes more and more accessible and the demand is growing, we will see even more commercial companies offering such drone surveys as a service, potentially with a different flavor each time targeting a specific industry or business domain.

- **Other types of cameras and assessment equipment.** As mentioned above, drones can fly easily and fast to, difficult to reach, assets and record high definition footage and images. The visual inspection is many times sufficient but not always enough as it does not give information or details on what is happening within the components of the asset.

In traditional inspections undertaken by humans, we can solve these issues intrusively (by accessing and inspecting the internal parts of the asset if it is safe to do so) or non-intrusively by utilizing thermal/infrared cameras, Lidar devices, gas detection systems, test and measurement equipment and many more. Given the increasing payload capacity of commercial drones, it is now possible to mount such devices and systems on a UAV and undertake this type of assessment remotely with the same results.

Utilizing drones to undertake this kind of inspections is very significant. When it comes to critical infrastructure where safety of the public is depending on the performance and functionality of overarching assets, the ability to schedule non-intrusive drone inspections frequently makes it possible for asset managers and transportation Engineers, to identify on time and with great accuracy any potential issues. Some of these issues could escalate to public hazards if they are not pinpointed and resolved early enough.

Another important point is that the use of drones for such inspections allows the Asset Authorities to accumulate enough temporal data of the asset conditions, undertake effective time series modeling and make better predictions on the asset life expectancy and expected performance.

- **Cognitive skills.** The continuously increasing processing power and decreasing size of super computers nowadays is exciting news when it comes to drones used in inspections. This technology revolution gives drones the ability to process video in real time using advanced vision algorithms and trained reference models with machine learning techniques.
It is becoming now possible to identify specific issues and conditions, found on transportation assets during inspections, by comparing the live video footage and images with a trained model of conditions and issues for the specific assets being inspected. In other words, a drone can now start identifying not only the type of mast on a street lighting pole and the barcode of the LED fitting, but also whether the mast has a corrosion issue and the light fixture cover is cracked. There are several companies and academic institutions experimenting with this technology and a lot of investment is going into building those reference models. It is fascinating to think that very shortly we will be able to place an inspection drone at the edge of a road corridor and collect it after some time when it will have surveyed, inspected and stored all issues and condition across all transportation assets laying across that corridor.

In the near future, the cognitive capabilities of drones could furthermore extend from processing video and audio to potentially identifying smell, taste and texture, and therefore able to decode and interpret everything, almost like a human brain.

- **Integral part of IoT infrastructure.** The increasing demand for better city and Infrastructure asset management, by Public Authorities, has led to the concept of Smart Cities which is now becoming a reality due to the evolution of the Internet of Things and Cloud computing.

  We see more and more transportation assets coming out of the manufacturers with built-in sensors and data transmitters that allow for remote telemetry, self-reporting and smart performance management. All this structured and un-structured data is contributing to what we nowadays like to call Big Data and eventually provides the Asset Owners with incredible insights that allow for really strategic and effective modeling.

  There are two points within this concept where Autonomous Inspection Drones can play a significant role. First of all, the cost to maintain live connection with all those sensors might be prohibiting for some Asset Owners. Imagine if the assets are hundreds of thousands and for every fixed number of assets, we required a communication controller, sim cards and monthly data fees with a mobile data service provider. In an attempt to reduce or even eliminate such costs, there is a growing number of business cases where drones equipped with appropriate sensor reading technology, fly through kilometers of road corridors and collect readings from all those sensors. The data is transmitted back to the operations centers for processing and modeling. This practice is definitely a much cheaper option, especially when there is really no need for real time data but for time-series data on a specific frequency.

  The second point, is that drones themselves can act as part of the sensor IoT infrastructure and not just as sensor data collectors and readers. The same way drones are nowadays equipped with sensors that prevent them from crashing or flying into obstacles, they can be equipped with sensors, robotic hands, motors and specialized electromechanical equipment to test, measure and assess on site. We have seen great examples of this concept in precision agriculture with drones monitoring levels of cultivation humidity on the field and subsequently spraying, through onboard devices, applicable nutrients and water with very high accuracy and precision.

  The transportation industry is actively looking into similar concepts where drones are not just monitoring the condition and operation but are gradually moving into “doing things” and interacting with the infrastructure when it is safe and wise to do so.

- **Augmentation.** Another technology that is growing rapidly, is augmented reality (AR). The concept of mixing reality with digital content and be able to comprehend interaction between them, is indeed impressive. The gaming and productivity software industry are embracing the technology fast.

  Conceptually, a drone camera is really not much different to an AR camera and the drone operator’s controller screen with bird’s eye view of what the drone sees, is also not that far from an AR
headset. What is missing is the fact that AR Headset Kits software is bringing reality and digital content together in a seamless manner. Although the development of such software for drones is still at early stages, there is a lot of interest by the Transportation Industry since it can potentially reduce inspection costs and time even more. Drones running on such software can “see” and understand in real time what is expected on the ground, across and around assets. In other words, drones will be flying in a mixed reality stereo model of the assets area, where the operator would be able to compare in real time what was designed in the office, what was installed for the as-built drawings and what is eventually being broadcasted by the drone camera in real time.

This technology is truly revolutionary and in combination with advanced real time analytics and cognitive computing, it is very likely that the future inspection drones will be called just inspectors.

Conclusion

The amount of recreational and commercial drones introduced to the market every year, is set to triple in the next four years. It is inevitable for industries like Transportation and for functions like the assessment and inspection of assets, to embrace drones even more. It is also expected to see a growing number of business applications and specialized inspection software for drones, reaching the market.

Drones are definitely solving issues around safety, cost and accessibility but the same time the industry needs even more standards and policies to ensure there are appropriate legal frameworks and Laws in place. It is only then that drones will be fully accepted widely as a professional tool to not only replace sometimes, most of what humans do in inspections, but to be established as the only way to do inspections and condition assessments.

As drones get increased battery life, more safety features, better hardware, more complex electromechanical parts & robot arms, more processing power on board and “brains” allowing them to recognize and do things “by will”, we are really moving into an era where drones could be really called autonomous because they would behave only as such.
ABSTRACT:
Autonomous vehicle technology has rapidly accelerated since 2004 when the Defense Advanced Research Projects Agency (DARPA) hosted the “DARPA Grand Challenge” to push the technology forward. It has grown from an idea that started on desert racecourses and in university labs to a technology available for retail purchase. Currently, many personal vehicles for sale include some level of autonomous and connected features and manufacturers are racing to deliver fully autonomous private vehicles. Nonetheless, widespread benefits for mobility, public safety, and equity may only be realized if the technologies are used to deliver safer, more efficient, and more effective public transit. This session will focus on presenting research and findings from case studies that illustrate how autonomous and connected vehicle technologies are being applied to improve transit service across multiple modes. Case studies will include:

- Tennessee Department of Transportation, Emerging Mobility Solutions Plan: Statewide plan identifying impacts of CAVs, developing policies and strategies, and defining implementation and pilot projects.
- Minnesota Department of Transportation, Autonomous Bus: Pilot project to test operation of full-size autonomous bus in cold weather conditions.
- Oklahoma City Autonomous Streetcar Feasibility: Study to determine how connected and autonomous vehicles can be used in a streetcar system scheduled to launch in 2018.
- Doha Automated People Mover Project: Implementing autonomous shuttles on an elevated dedicated guideway to provide local circulation and first/last mile connections to other transit.

The session will provide attendees with key takeaways on how CAVs can improve transit, including policy, pilot projects, and integration into existing fixed-guideway systems.
Leveraging Connected and Autonomous Vehicle Benefits for Public Transit

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1. INTRODUCTION

Autonomous vehicle technology has rapidly accelerated since 2004 when the Defense Advanced Research Projects Agency (DARPA) hosted the “DARPA Grand Challenge” to push the technology forward. It has grown from an idea that started on desert racecourses and in university labs to a technology available for retail purchase. Currently, many personal vehicles for sale include some level of autonomous and connected features and manufacturers are racing to deliver fully autonomous private vehicles. Nonetheless, widespread benefits for mobility, public safety, and equity may only be realized if the technologies are used to deliver safer, more efficient, and more effective public transit.

The purpose of this paper is to demonstrate how connected and autonomous vehicle technologies can be leveraged for public transit benefits. Section 2 of this paper provides an overview of existing technologies being developed for connected and autonomous vehicles that have applicability for transit uses, as well as potential impacts of these technologies. Section 3 presents research and findings from case studies that illustrate how autonomous and connected vehicle technologies are being applied to improve transit service across multiple modes. Section 4 concludes the paper with recommended approaches public agencies, stakeholders, and partners can take now to leverage connected and autonomous vehicles for public transit benefits.

2. CONNECTED AND AUTONOMOUS TECHNOLOGIES OVERVIEW

2.1. Definition of Connected and Autonomous Vehicles

Connected vehicles use technology to communicate with nearby vehicles and infrastructure. These vehicles use this information to enable crash prevention by providing a 360-degree awareness to provide vehicle operators with information they might otherwise be unable to see.

As defined by the United States Department of Transportation, automated vehicles are those in which at least some aspect of safety-critical control function (e.g. steering, throttle, or braking) occurs without direct driver input. Automated vehicles may be autonomous (i.e. use only vehicle sensors) or may be connected (i.e. use communications systems in which cars and roadside infrastructure communicate wirelessly).

Connected and autonomous vehicles (CAV) rely on a combination of mapping, sensor, and communication technologies, including, but not limited to, global navigation satellite systems (GNSS), radar, stereo cameras, LiDAR, cellular and direct short range communication (DSRC).

CAVs have three primary sources of input in standard operation:

- Local sensor detection. These vehicle-mounted sensors detect environmental or other non-connected obstacles, including pedestrians, cars and miscellaneous hazards. The sensor hardware can include an array of ultrasonic distance sensors, to a LiDAR system.
- Vehicle-to-Vehicle (V2V) and Vehicle-to-Pedestrian (V2P). These are smart systems used on a local scale. Any entity with this technology receives and transmits its location data and destination to help optimize local flow.
- Vehicle-to-Infrastructure (V2I). Similar to V2V and V2P, but used on a larger scale for planning regional traffic flow based on current demand.

The following section describes CAV technologies applicable to transit.

2.2. Applicable CAV Technologies

2.2.1. Geolocation

It is important for all automated vehicles to understand where they are in the world. This helps a vehicle determine what rules to apply given its location. For example, location will determine speed limits. Typically, nearly every electronic device that has to locate itself in the real world utilizes a global positioning system (GPS). Communications with geostationary satellites allow a device to triangulate to determine its location on earth. However, GPS has reliability issue when a receiver cannot be in line-of-site with at least three satellites. This can happen in cities or “urban canyons” where tall buildings and other infrastructure can interfere with communications. To help address this issue, differential GPS is sometimes utilized to supplement the satellite system. This is done through the installation of terrestrial positioning base stations nearby the device needing to be located. These extra signals allow the device to determine its location more accurately.
Automated vehicles are now utilizing a combination of GPS and dead reckoning to geolocate. Dead reckoning is done by first creating or programming a digital terrain map for the vehicle. This can be done through training runs if the course is fixed. Once the map is programmed into the vehicle, the vehicle can determine where it is at any given time through accelerometer readings which provide trajectory, speed, and direction information to the vehicle.

2.2.2. Collision Avoidance

Without a human at the controls to prevent a vehicle from colliding, a robust sensor network is needed to provide information to the vehicle on when to speed up, slow down or stop. Automated vehicles use a few different types of technologies to prevent collisions. Oftentimes vehicles use any combination of these.

![LiDAR Sensor](Source: Delphi)

LiDAR sensor devices are most commonly used. These devices are installed within the vehicle body or sometimes as an ancillary device outside of the vehicle. This device works similarly to radar except instead of using radio waves, it uses light waves in the invisible light section. The device determines the distance of objects by measuring the time it takes for the light to bounce back from the object. Sometimes as many as twelve of these devices are installed on a vehicle to provide the ability to see objects in all directions. These devices create a rich, detailed data set that represents the dynamic and stationary elements in the field of view.

Another technology that has advanced significantly in the past three years is the use of high definition cameras with video analytic technology. The video analytic engines now coupled with cameras allow for vehicles to interpret objects as they appear in their field of view. This provides information on the classification of the object, like determining if it is a pedestrian, bicyclist, car, or construction pylon.

Lastly, advancements in connected vehicle communications have resulted in collision avoidance systems for intersections – otherwise known as Cooperative Intersection Collision Avoidance Systems (CICAS). Intersection collision avoidance systems use both vehicle-based and infrastructure-based technologies to give information to a vehicle approaching an intersection the state of that intersection. These systems have the potential to provide warnings or control of a vehicle as it approaches and moves through an intersection. A CICAS consists of system sensors and processors in the vehicle and an infrastructure interface with signal system. A Dedicated Short-Range Communications (DSRC) radio would be utilized to communicate warnings and data between the infrastructure and equipped vehicles.

![Vehicle-Infrastructure Communication](Source: USDOT)
2.2.3. Signalized Intersection

While most automated vehicles used in public spaces today do not communicate or operate with a signalized intersection, the automated transit vehicle manufacturers interviewed for this study indicated that they will utilize DSRC as a primary means of knowing when to stop or transverse through an intersection. This will allow a vehicle to understand what the signal, phase, and timing (SPaT) is at any given time. The manufacturers interviewed for this study indicated that they supplement this primary means of knowing the state of the intersection with video recognition, which can allow the vehicle to see the color of the light as well as determine if there is anyone running a light.

![Signalized Intersection](Image)

Figure 3. Signalized Intersection. (Source: USDOT)

2.2.4. Artificial Intelligence

Supplemental to the sensor and communications networks mentioned above, some automated vehicles are being implemented with machine learning and cognitive intelligence. Machine learning allows a vehicle to remember previous activities and utilize this memory in decision making when encountering new situations. Cognitive intelligence provides an inference engine to the vehicle so that it can make more human-like decisions. Due to the costs and early development stages of artificial intelligence, it is recommended that any artificial intelligence be limited to driving functions addressing unforeseen visual cues.

2.3. Potential Impacts of CAVs to Transit and Mobility

Connected and autonomous vehicle technologies have the potential to result in significant safety, mobility, and sustainability benefits. For example:

- **Safety:** In the U.S., more than 30,000 people die in traffic-related deaths every year. According to the National Highway Traffic Safety Administration (NHTSA), 94% of accidents are related to human error. Introducing connected and autonomous vehicles can eliminate most of the human error and prevent a majority of accidents.
- **Mobility:** Autonomous vehicle technology such as truck platooning, adaptive cruise control, and real-time route optimization may significantly mitigate congestions on local roads and highways. Connected and autonomous vehicle technologies could also be used to provide more efficient, cost-effective transit service, or to introduce new services.
- **Connectivity and Equity:** Autonomous shuttles can connect low density areas and underserved communities with high capacity, high quality transit networks. Autonomous vehicles can also provide mobility options for people with disabilities that currently prevent them from driving.
- **Sustainability:** Most autonomous vehicles are assumed to also be electric vehicles, thereby reducing reliance on fossil fuels. Autonomous and connected vehicle technologies may also increase highway speeds and result in more fuel-efficient driving with smoother acceleration and deceleration, contributing to decreased energy consumption.
- **Economic Impact:** The energy and time saving brought by autonomous vehicles can generate net economic benefits between $5 to $7.5 trillion (2015 dollars) over the next 30 years in the U.S.

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• Flexibility: Without having to physically drive the car, multitasking while traveling long-distance is enabled.
• Land Use: Vehicles that park themselves or have little to no need to park in certain areas may free up valuable land currently dedicated to parking for other public benefits. Autonomous vehicles may also be able to operate in narrower road lanes, enabling new uses for the public right-of-way.

The timeline of this technology hitting the market ranges. The Boston Consulting Group (BCG) has published a study on autonomous vehicles adoption in the future, predicting 25% of new vehicle sales by 2035 will be fully autonomous.\(^3\) Based on an analysis of previous technologies (automatic transmission, airbags, hybrid vehicles, navigation systems) the Victoria Transport Policy Institute estimates it won’t be until the 2060s until most vehicles on the road are autonomous.\(^4\) Over the next several decades, the vehicle fleet will likely be a mix of fully autonomous, some-autonomous, and driver-operated vehicles.

However while there are many potential benefits, the real impact of autonomous vehicles will be shaped by how they are planned for today. The International Association of Public Transport, or UITP, notes that the benefits of autonomous vehicles may only be achieved if the vehicles are used in conjunction with:

• A high capacity core network with fixed line services,
• As feeders to public transport stations,
• As area based on-demand mini-buses, or
• As autonomous car sharing vehicles.

Absent policies, regulations, or incentives to encourage the shared use of autonomous vehicles, UITP warns of a potential “swarm” of autonomous vehicles as robo-taxis and on-demand shuttles.\(^5\) Privately owned autonomous vehicles may also result in increased vehicle-miles traveled, as individuals tolerate more and longer trips during which they can be productive or relax rather than drive; as those who couldn’t previously drive could become riders (for example children or older adults who no longer drive); and as the concept of zero-occupancy vehicles becomes a reality (vehicles circulating with no human occupant). Finally, autonomous vehicles may promote sprawling land use development as people may be willing to tolerate longer commutes.

3. CASE STUDIES

Connected and autonomous vehicle technologies have the potential to improve the efficiency and safety of transit services, as well as to provide new service and first/last mile connectivity to existing transit networks. Several departments of transportation and local transit agencies are currently exploring the use of connected and autonomous vehicle technologies for transit applications. This section describes case studies from around the globe illustrating the various approaches agencies are taking to incorporate CAV technologies for transit, including applications for rail and bus, as well as incorporation in long-range transportation plans and policies.

3.1. Vitry-sur-Seine, France – Alstom & RATP

In Vitry-sur-Seine, France, the RATP (Autonomous Operator of Parisian Transports) has been testing an Alstom tram retrofitted with LiDAR and other technologies from EasyMile, a French autonomous technology company known for their autonomous shuttle vehicles. The research and development project is using an existing vehicle to test the feasibility of having autonomous driving on the existing T7 tramway line in the suburbs of Paris. The project is being done in a maintenance depot without passengers, and the first phase of initial testing was completed over a six month period including installation, testing, and validation. The project tested the presence of obstacles including bicycles, and the vehicle’s ability to recognize, brake emergency brake, and decelerate in the presence of obstacles. The tram can detect obstacles and apply brakes as appropriate, including emergency brakes when necessary. The tram has also been programmed to find its stabling point in the yard. Because the testing was done in a depot rather than a public road, no special permits or regulatory approvals were needed, however this would need to be addressed during the next phase of testing. The initial testing phase was completed and announced in April 2017. According to an interview conducted with Alstom in June 2017, the project team was in the process of scoping out the next phase of the project (The exact next steps and timeline were not yet decided at the time of the interview. The project cost is confidential).

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3.2. Zhuzhou, China – CRRC Corporation Limited (CRRC)

CRRC’s “railless train” is a fully-autonomous 300-seat electric bus with rubber tires. It is bidirectional and has a maximum range of 25 miles and a top speed of 43.5 miles per hour. Because it does not travel on a track, it can be rerouted at will by a remote command in case of traffic or other road obstacles. CRRC plans to open a four mile demonstration route in Zhuzhou, China in 2018.  

3.3. Washington State, USA – Transit Agencies of Richland, Vancouver, Everett, Olympia, Bremerton, Tacoma, Spokane & Seattle

Mobileye partnered with eight Washington State transit agencies to outfit a total of 38 buses with their Shield+ system for collision monitoring and warning. Several metrics were examined during the trial:

- Ease of retrofit – by the end of the trial, a system could be outfitted on any vehicle in 16 person-hours.
- Safety – no pedestrian or bicycle collisions occurred during the trial.
- Operator satisfaction – 63% of operators found the system distracting, with many noting false warning alerts when approaching a stop with passengers waiting. 67% said they had a negative overall experience with the Shield+ system.

The study recommends improving the system and adding contextual filters, for example, removing alerts for pedestrians at a known bus stop.

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3.4. Tampa, Florida, USA – TECO Line Streetcar System

As part of the Tampa Connected Vehicle Pilot, existing streetcars are being equipped with connected vehicle technology that will allow the streetcars to “see” any connected vehicle or pedestrian, and alert the driver to stop when there is a conflict. The system will not control the streetcar, but simply act as an early warning for the operator to react as necessary.  

- The pilot will install connected vehicle systems on ten streetcars. The systems can receive signals from pedestrian sensors outfitted in crosswalks, or personal mobile devices, as well as connected vehicle systems.
- The Pilot will deploy 1,600 private vehicles with the connected systems, ten buses, ten streetcars (retrofit Heritage streetcars), and 500 participating pedestrians.
- The cost for the pilot (including vehicles, buses, and streetcars) is being funded by a $17 million contract with the USDOT and a $4 million contribution from the Tampa Hillsborough Expressway Authority from toll revenue.
- The schedule includes three phases: concept development (September 2015-August 2016), design, implementation, and testing (September 2016-april 2018), and operation and maintenance (May 2018-December 2019).
- Equipment for the Connected Vehicle Pilot is being provided by several sources. Siemens is developing the roadside units that will communicate with the connected vehicles and the City’s Transportation Management Center via DSRC. Brandmotion, the vehicle systems integration partner, is working with Savari, Commsignia, and SiriusXM to supply onboard units that will be installed in vehicles. These units will display safety messages.

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3.5. Minnesota, USA – Autonomous Bus Pilot Project

The Minnesota Department of Transportation (MnDOT) is currently conducting an Autonomous Bus Pilot Project. This project will explore the use of an autonomous bus in cold weather climates. The project is specifically focused on a mass-transit application for autonomous buses. As noted by MnDOT, while many mid- to high-end personal vehicles include some level of connected and autonomous features, little research is being done for transit applications. The MnDOT Autonomous Bus Pilot Project seeks to address this by soliciting technology partners and conducting a controlled demonstration to identify the challenges and strategies to operate autonomous buses on the State transportation system.8

3.6. Oklahoma City, Oklahoma, USA – Autonomous Streetcar Feasibility Study

Oklahoma City is currently determining the feasibility of applying connected and autonomous vehicle technologies to the Oklahoma City Streetcar. The streetcar is being designed to run rails flush with the street and will operate on the street mixed with other transit and passenger vehicles, pedestrians, and bicyclists.9 The purpose of the Oklahoma City Autonomous Streetcar Feasibility Study is to determine if and what connected and autonomous vehicle technologies can be incorporated into the streetcar to improve efficiency and safety.

3.7. Tennessee, USA – Tennessee Department of Transportation, Emerging Mobility Solutions Plan

Through the Emerging Mobility Solutions Plan, the Tennessee Department of Transportation (TDOT) Long Range Planning Division is charting a bold new path that embraces and accelerates the use of transportation technology to solve increasingly complex and ambitious transportation challenges. To best harness the transformative role technology (including connected and automated vehicles), will play in the future, the document assesses the applicability of recent transportation technologies and trends to TDOT’s specific needs, and incorporates them into the Department’s 25-year Long-Range Transportation Policy Plan.

For years, TDOT has been working with federal and local governments to integrate new Intelligent Transportation Systems (ITS) technologies into the existing transportation system. Doing so has helped emergency response teams react quicker, and has also kept the general public up to date on current traffic conditions across the State; but this is only the beginning. As discussed in the 25-year Policy Plan, TDOT’s Vision and Guiding Principles were established to lead Tennessee to the ultimate goal of “…providing the best multimodal transportation system in the nation.” Furthermore, TDOT’s 10-year Strategic Investment Plan has identified the need to emphasize transportation reliability, availability, and accessibility through various improvements involving interstate modernization, multi-modal connectivity, and ITS.

With the Emerging Mobility Solutions Plan, TDOT is further advancing the State’s comprehension and use of connected and automated vehicle technology, defining to what extent TDOT can use this technology to positively impact future transportation plans of the not-to-distant future, and outlining the steps needed to ensure a smooth and seamless transition.

3.8. Doha, Qatar – Doha Automated People Mover Project

The Ministry of Transportation and Communications (Mo TC) - Technical Affairs Department is completing a feasibility for implementing an Automatic People Mover (APM) within the West Bay of Doha, Qatar.

This Urban Center is considered the most prominent and busiest district of Doha. It has the potential and opportunity to become a vibrant, livable, sustainable community. Doha’s urban communities depend highly on motorized movement and areas of the city are becoming congested, negatively impacting the living environment. In response, the MoTC are proposing to introduce an APM in the District to create spaces that are accessible, walkable, and pedestrian oriented with a highly integrated transit system.

The feasibility study suggests schemes and solutions to implement the APM in West Bay. The scheme is composed of 9 loops and 37 stations that run over a 10Km track. The system is anticipated to accommodate for 5,000 passengers per hour. The strategy is to ensure a loop system is implemented to ensure efficiency and fluidity of spaces.

4. CONCLUSION: RECOMMENDED APPROACHES TO LEVERAGE CAVS FOR PUBLIC TRANSIT BENEFITS

The advent of CAVs is causing vehicle makers to rethink their roles; transportation agencies to imagine new ways of providing service; and urban planners to consider new uses for parking lots, curbs, and other urban spaces historically used by vehicles.

Autonomous vehicles have the potential to deliver an array of benefits, but their integration into our cities and communities must be thoughtfully planned. As demonstrated by the case studies summarized herein, transit agencies are using pilots, feasibility studies, demonstration projects, and long-range planning to forecast, develop and meet federal, regional, and local safety, mobility, and environmental goals using CAV technologies. Lessons-learned from the case studies points to these key ideas:

- Technology is a tool, not a solution in itself. The exciting thing about CAVs is not that cars will drive themselves, but that great advances can be achieved for safety and mobility. Rather than focusing on what agencies can do to accommodate CAVs, they are focusing on how CAVs can advance our mobility, equity, and safety goals and address specific local needs.
- Partnerships and pilots will pave the way. Recent advances in transportation technology have brought new players into the field, from ride sourcing to software companies. Partnering on pilot projects with new mobility service providers can allow transit agencies to test solutions and move technology forward in a way that best serves local needs.
- Planning is imperative. While autonomous vehicles will deliver safety benefits, many of their other impacts are still largely speculative. Now is the time for scenario planning to bookend potential future projects, and to evaluate how present-day policy and infrastructure decisions can help shape CAV deployment to best meet mobility goals.

The possibility of an automated future is promising, but CAVs are not a panacea for our transportation challenges, nor should they be. In high-capacity corridors, transit solutions, like rail, will best serve demand. However, along with bicycle and pedestrian infrastructure, CAVs can be a useful tool to extend the reach of high-capacity transit and serve lower density areas beyond the reach of these networks. As part of a multi-modal suite of solutions, CAVs can expand transportation choices and better serve riders. According to some estimates, by 2060 almost all cars on the road will be CAVs. By advancing CAV implementation through long-range planning and feasibility analysis, strategic partnerships, and pilot project design and deployment, transit agencies will be in the best position to leverage CAVs for the benefit of public transport and mobility.
CITATIONS


How Machine and Deep Learning Technologies will Revolutionize Intelligent Transport Systems and Smart Cities

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INTRODUCTION

Machine learning and deep learning are emerging technologies that will significantly impact the intelligent transport systems and Smart Cities worldwide. As a matter of fact, they will impact many facets of our lives not just the transportation industry. What makes machine learning unique is its incredible power to process far more data than a single human could ever handle. These technologies will bring to life new solutions and application not possible in the past.

This paper intends to shed light on these technologies, and how they could be used in the transportation industry and what types of challenges and benefits we could realize from them.

WHAT IS MACHINE LEARNING AND DEEP LEARNING?

Machine learning is an Artificial Intelligence (AI) application were computers can provide enhanced solutions from learning and experience in analyzing various forms of data. With the ever-increasing amount of data that is generated by the internet and Internet of Things (IoT) it is not very difficult to teach computers about each world situation. It is better to teach computers to learn how to think like humans.

The development of neural networks has been key to teaching computers to think and understand the world in the way we do, while retaining the innate advantages they hold over humans such as speed, accuracy and lack of bias. A neural network is a system designed to work by classifying information in the same way a human brain does. It can be taught to recognize, images and classify them per certain elements or features they contain. It works on a system of probability – based on the data it is given as input, it can make decisions or predictions with a degree of certainty. The addition of a feedback loop enables “learning” – by sensing or being told whether its decisions are right or wrong, it modifies the approach it takes in the future. Figure 1 shows a typical machine learning process.

Machine learning has the promise of automating routine tasks and offering creative insight for various industries including transportation, banking, healthcare, retail, and manufacturing.

Deep learning takes machine learning a step higher in the development of “intelligent solutions or outcomes” by machines. It is the natural progression of machine learning as technology evolves and as research and development discovers new algorithms and techniques. This paper is not intended to be a primer on either machine learning or deep learning techniques but it is intended to discuss some of the many practical uses of these technologies in the transportation domain.

TRANSPORT SOLUTIONS USING MACHINE LEARNING

Machine learning techniques make it possible to derive patterns and models from large volumes of data as well as handle high-dimensional data, where each data sample consists of many data points. Big-data platforms leverage
distributed file systems and parallel computing to enable fast processing of data, enabling both real-time and predictive analysis. Improved visualization and simulation tools have also greatly expanded the utility of all these new data, making them easier to use for predictive modeling and making the graphical outputs more readily understandable and even more insightful.

There are many applications of machine learning and other artificial intelligence technologies that has the potential to impact and transform the transportation industry. These applications span many disciplines within the transportation industry such traffic management for roads, autonomous vehicles, connected vehicles, fleet management commercial freight, logistics, operations, asset management and inspection, etc. This paper we will focus on only three transport areas that could see the biggest transformation due advances in machine learning and artificial intelligence technologies:

A. Roadway Traffic Management
B. Self-Driving Transport
C. Asset Management

A. Roadway Traffic Management

Roadway traffic management has not seen many breakthrough advances in the past 15 years. With the advent of AI and specifically machine learning algorithms and applications that is set to change how we go about managing congestion on our roadways and creating safer roads. Within the roadway traffic management areas there are several key areas were machine learning can provide significant tools that were not possible before. These areas include but limited to:

1. **Network-level operations and management.** Real-time including predictive analytics for the proper network level management and not just roadway level management. Historically and limited by human and computing limitations, we were unable to analyze large volumes of data (big data) feeds from multiple sources and make timely decisions and take actions across the entire network. The ability of today’s computing platforms to use sophisticated machine learning algorithms to troll, analyze large amounts of various data types and formats based on past experience and learning offers a myriad of solutions that were not possible in the past

2. **Traveler guidance.** Traffic systems can display the current location and status of all incidents from multiple sources, including social media. Ultimately incident duration prediction from machine learning can estimate the duration from response activation to clearance time

3. **Large scale/real-time simulations.** The capability now exists for making sense of large traffic data streams in real time, as well as supporting large-scale traffic simulation and traffic forecasts of various durations. The future of traffic simulation is no longer restricted to fixed empirical equations but considers past experiences and historical data and the many probabilities of their occurrence.

4. **Transport management Centers.** Drones utilizing machine vision with machine learning algorithms will become an integral part of future transport centers. These drones will provide real-time tools to optimize the management of incidents on our roadways.

B. Self-Driving Transport

Machine learning is about: pattern recognition. Pattern recognition is part of what makes humans intelligent: it’s part of what enables us to learn from experience or transfer knowledge from one domain to another. Machine learning gives that intelligence to machines and the possible application in the field of self-driving vehicles.

Self-driving vehicles has the potential to solve many problems facing us today such as congestion and safety issues. Most autonomous vehicles are equipped with many smart camera systems, all connected to a computing platform focused on safe and efficient driving, which is itself instantly connected to every other vehicle on the road to learn from their experiences and communicate with them at the very high speed. When you combine that singular focus, that array of sensors, and fleet-level communication and learning, has the potential to create the world’s safest, most efficient vehicle. To do that, you need to make it intelligent by equipping it with the ability to recognize pedestrians, stop lights, street signs, and other objects. Its communication with other vehicles needs to come with rules for decision-making and planning. It must understand the human context of navigation, like ‘dropping off’ and ‘following.’ It must be able to do hundreds of tasks that have previously been relegated to human intelligence; in other words,
it must have artificial intelligence, and this is where machine vision, machine learning, and deep learning all contribute towards this intelligence.

Therefore, machine learning has the potential to save lives by avoiding accidents due to human errors attributed to inattention, fatigue, and/or impairment. This is accomplished by teaching machines to recognize and take appropriate actions when encountering various objects or features on the road such as other vehicles, pedestrians, traffic signs, road marking, road geometry, etc.

In addition, machine learning can create efficient carpooling and other mobility services for self-driving vehicles.

Machine learning in connected and autonomous vehicles can:
- Learn where you parked your car,
- Learn destinations that you recently visited, including your home and work locations,
- Learn frequent routes that you take regularly,
- Predict your trips and advise you when you should leave.

C. Asset Management

Machine learning and specifically machine vision is already changing the way we collect data and manage this data to better manage our assets. These assets could be roadway furniture such as signs, marking, signals, lights, buildings, power lines or other types of utilities. The possibilities are endless and cost savings are significant. Some of the key areas where machine learning is already transforming the way we could better manage our assets include but not limited to:

A. 3D mapping using machine vision and machine learning enabled drones
Drones are deployed and flown over transportation infrastructure assets. They are equipped with very high resolution cameras and incredibly fast GPUs on board. The outcomes are high quality point cloud data files and even centimeter accuracy 3D models (using specialized post processing software) that allow the Asset Managers to identify quickly and with great accuracy potential faults, asset failures, health & safety issues and asset non-performance proof.

B. Remote inspections using machine vision and learning enabled drones
Extending the concept behind the previous point, drones can nowadays be deployed beyond visual line of sight through advanced command and flight control UAV centers and transmit data in real time from many kilometers away using cutting edge communication protocols and advanced mobile technology infrastructure. As such Asset Managers, can operate fleets of remote drone inspectors that are deployed upon certain event and triggers or on demand by remote operations staff.

C. Data collection using feature recognition and extraction using machine vision and learning
The use of deep learning in video analytics applications, allows the extraction of targeted information from live or pre-recorded footage. Deep learning allows for indexing and classification of shapes, colors, driving behavior patterns, vehicles trajectories and change of direction, speed and many more. In transportation, this presents Professionals in this field with great opportunities to count, measure and monitor the mobility on the streets a lot faster and with impressive accuracy.

D. Preventive maintenance using predictive analytics. As an example, sensors currently allow transit system operators to remotely monitor rolling stock equipment vibration (a performance indicator), video cameras, HVAC equipment, temperature, humidity, system alerts, and fault warnings. These systems can enable a risk-based approach to preventive maintenance. In addition, predictive analytics have the potential to improve defect investigation for all types of vehicles.
CHALLENGES AND OPPORTUNITIES TO IMPLEMENTING THESE TECHNOLOGIES IN THE TRANSPORTATION INDUSTRY

As with any new technology there will be challenges to its implementation. The biggest challenges in the industry include but not limited to:

Challenges:

A. Lack of understanding of the transportation professional about machine learning and deep learning and how they might help in creating new solutions that were not possible in the past.
B. Lack of subject matter experts in machine learning and deep learning who also understand the transportation industry and the problems needing new approaches and solutions.
C. The pace of advancement specifically in the field of deep learning makes it difficult to implement unless brought to the market by business at the reasonable cost.
D. The hesitancy of certain transportation professionals to try or adopt emerging technologies

Opportunities:

A. The interest in connected and autonomous vehicles by “non-traditional” corporation to the transportation industry is creating a new opportunity for new thinking and investments. Companies like Intel, Nvidia, Apple, Tesla, Google, in addition to traditional auto manufacturers are investing billions of dollars using their know how in machine learning and deep learning. This will allow new technologies to come to market faster and that could be leveraged by transportation professionals.
B. Many universities and research institutions are key to where most of these technologies are created. And most of these institutions are willing and eager to partner with both the public and the private sectors.
C. The proliferation and adoption of Cloud Computing is enabling small and large companies and agencies to deploy and test machine and deep learning based solutions

CONCLUSION

Advances in technology such as artificial intelligence, machine learning, and deep learning are here to stay. As a matter of fact, they will impact every aspect of our lives in the coming years and the transportation industry is no exception. There is a great opportunity to leverage the emerging developments being created by new companies and businesses who traditionally did not play in the transportation space to enhance safety and mobility of our transportation systems.

As consumers, machine and deep learning technologies will change the way we use and buy transportation products and services in the next decade and beyond.

As transportation industry professionals machine and deep learning technologies will empower us by offering tools to solve complex problems, save lives, and reduce congestions, and provide customized solutions and choices to our clients.
In late 2015, the United States Department of Transportation launched the Smart City Challenge, asking mid-sized cities across the U.S. to develop ideas for an integrated smart transportation system that would use data, applications, and technology to help people and goods move more quickly, cheaply, and efficiently. The Smart City Challenge identified 12 vision elements defining a smart transportation system. ConSysTec specializes in development of Intelligent Transportation System (ITS) Architectures and Standards, and has significant experience with standards-based specification development, and has been called upon to conduct standards gap-analyses for the U.S. Connected Vehicle and Data Capture and Management programs. ConSysTec also consults with municipal governments and authorities regarding Smart City deployments.

This paper documents ConSysTec’s research into available ITS Standards, both U.S. and European, necessary for deployment of open Smart City services. This paper walks through each of the 12 vision elements, identifying applicable open data and communications standards, and potential gaps in standards. The identified gaps should be used to create awareness of risks in deployment of smart city services related to the gaps, and guidance to standards development organizations on where open standards development investments should be considered.

INTRODUCTION

The U.S. DOT’s Smart City Challenge Vision Elements provide a roadmap for assessing the applicability of architecture and standards to enable interoperable transportation solutions for Smart City projects.

The Vision Elements are described in the figure below.
Figure 1. Smart City Challenge Vision Elements

What follows is an assessment of each vision element, including:

- **Description.** A summary of the vision element drawn from the Smart City Challenge Request for Proposal.
- **Suitable Projects.** Example projects that may fulfill the needs of the Smart City Challenge Vision Element.
- **Applicable Standards.** An assessment drawn from the authors’ experience with development of communications standards for Intelligent Transportation Systems. The authors indicate where additional research may be needed.

**Vision Element #1: Urban Automation.**

**Description**
Automated transportation offers possibilities for enhancing safety, mobility, accessibility, equity, and the environment. The Smart City can provide automated transportation applications and systems for the movement of goods and people.

**Suitable Projects**
- Self-driving vehicles;
- Self-driving shuttles operating at low speeds to enable mobility options for first/last mile travel to local destinations and access to public transportation;
- Fully automated trucks and buses used in intermodal facilities, such as ports, depots, and maintenance facilities to improve driver and vehicle efficiencies; and
- Driver-assisted automation to reduce congestion and localized pollution and smog.

**Applicable Standards**
- Requires further investigation.

**Vision Element #2: Connected Vehicles.**
Description
Connected vehicles use vehicle-to-vehicle (V2V) and vehicle-to-infrastructure (V2I) communications to provide connectivity that will enable safety, mobility, and environmental applications. Connected vehicle technologies allow vehicles to send and receive information about their movements in the network – offering cities unprecedented opportunities to provide more responsive and efficient mobility solutions in real-time and in the long term. Data derived from connected vehicles provide insights to transportation operators, help to understand demand, and assist in predicting and responding to movements around a city. In deploying connected vehicle and infrastructure services, Smart Cities may seek to integrate a variety of commercially available communication technologies including cellular, satellite, Wi-Fi, and Dedicated Short Range Communication (DSRC) technology.

Suitable Projects
- Connected vehicle applications including safety, data collection from mobile sources, and environmental quality

Applicable Standards
- Society of Automotive Engineers (SAE) DSRC
- NTCIP Standard for Actuated Signal Controllers, provides communication between signal controllers and vehicles for safety applications

Vision Element #3: Intelligent, Sensor-Based Infrastructure.

Description
Smart cities use intelligent infrastructure that allow sensors to collect and report real-time data to inform transportation-related operations and performance and trends of a city. These data allow city operators to evaluate how the city is operating and how to enhance the operation of facilities, systems, services, and information generated for the public. Intelligent infrastructure includes sensors that collect traffic, pedestrian, bicyclist, environmental data, and other information available throughout the city.

A successful Smart City will integrate these data with existing transportation data and operations, allowing the city to improve operations of the transportation network.

Additionally, infrastructure could be used to monitor transportation assets to improve infrastructure management, reduce maintenance costs, prioritize investment decisions, and ensure a state of good repair. Where possible, a Smart City will make these data accessible to a broader ecosystem of developers to enable new research and applications. Smart Cities should leverage existing infrastructure investments, including sensors operated by other public sector agencies, academia, the private sector, and personal mobile devices.

Suitable Projects
- Real-time infrastructure monitoring
- Data collection for transportation archives

Applicable Standards
- Internet of Things Standards from Internet Engineering Task Force (IETF)
- World Wide Web Consortium (W3C) Standards
- NTCIP Standards for Roadway Equipment, including:
  - Actuated Signal Controller (sensor interfaces)
  - Environmental Sensor Stations
  - Closed-Circuit Television Cameras
  - Data Collection and Monitoring Equipment
  - Transportation Sensor Systems
  - Electrical and Lighting Management Systems
**Vision Element #4: Urban Analytics.**

**Description**
This vision element includes platforms to understand and analyze data to address complex urban challenges (e.g., personal safety and mobility, network efficiency, and environmental sustainability) and to measure the performance of a transportation network. Data-rich environments are increasingly able to share, use, and leverage previously unavailable datasets to address complex urban problems and improve current operations and capabilities. Urban analytics create value from the data that is collected from connected vehicles, connected citizens, and sensors throughout a city or available from the Internet using information generated by private companies.

Analytics can be used to predict future conditions and the potential benefits of implementing different operational strategies, control plans and response plans coordinated among agencies and service providers. One example might be an application of travel demand management that also factors in environmental and energy consumption as part of the optimization — providing more context to citizens’ personalized recommendations. To do so, transportation-related performance measures and evaluation are needed to quantify the intended and measured impact of all proposed solutions on personal safety and mobility, network efficiency, and environmental sustainability, representing the priorities of this challenge.

For example, performance measurement may indicate greater access to jobs and services; reduction in congestion and delays; increase in transit, walking, or cycling; a reduction in crashes, injuries, and or fatalities; improved incident response and clearance times; and reductions in emissions. In a Smart City, these performance measure should be made publicly available as open data.

**Suitable Projects**
- Transportation demand management
- Transportation predictive analytics
- Transportation archive systems
- Transportation performance measurement

**Applicable Standards**
- Institute of Transportation Engineers (ITE) Traffic Management Data Dictionary
- The Transport Protocol Experts Group (TPEG) is a data protocol suite for traffic and travel related information. TPEG can be carried over different transmission media (bearers), such as digital broadcast or cellular networks (wireless Internet).

**Vision Element #5: User-Focused Mobility Services and Choices.**

**Description**
Smart cities support sustainable mobility using traveler-oriented strategies that deliver innovative solutions across all transportation modes, including transit, bicycling, electric vehicles, and shared use mobility services, to improve the mobility of all travelers, including older travelers and people with disabilities. A major component includes advanced traveler information systems that provide real-time traffic, transit, parking, and other transportation-related information to travelers. Shared-use transportation has grown tremendously in recent years with the increase in smartphone applications. Open data and technology enable the efficient coordination, use, and management of all mobility services in the system. From the user’s perspective, travel choices are simplified through open data and communications technology that provides personalized information — including traveler information, travel options, and integrated mobile payment — directly to the user. Smart cities leverage mobility services, integrated transit networks and operations, real-time data, connected travelers, and cooperative ITS.

**Applicable Standards**
- General Transit Feed Specification (GTFS) provides common formats for electronic schedule data
• GTFS-realtime provides for communication of vehicle location, estimated arrival/departure information, and traveler alerts.
• Service Interface for Real Time Information (SIRI), part of the Transport Protocol Experts Group (TPEG), provides for vehicle location, journey times, arrival/departure, and alerts
• NTCIP Standard for Transit Signal Priority, provides standardized interface between buses and signal controllers

Suitable Projects

• Traveler Information, Travel Time Reliability
• Bus Rapid Transit (BRT)
• Mobility on demand operations
• Rideshare systems

Vision Element #6: Urban Delivery and Logistics.

Description
This vision element includes solutions that support efficient goods movement through use of data or technology to create opportunities for a more efficient supply chain approach that delivers safer logistics management, improved on-time pickups and delivery, improved travel time reliability, reduced energy use, and reduced labor and vehicle maintenance costs. As populations increase and urbanization continues, cities need to identify innovative ways to effectively and efficiently move goods – including food, energy, and manufactured goods – into and throughout cities. The Smart City may consider improving urban goods movements by including freight-specific information exchanges that enable dynamic travel planning to improve freight movement efficiency, including load matching and drayage operations.

Applicable Standards

• Requires further research.

Suitable Projects

• Port operations
• Border crossings
• Real-time freight logistics monitoring (schedule maintenance and routing)
• Real-time monitoring of freight movement cost

Vision Element #7: Strategic Business Models and Partnering Opportunities.

Description
Opportunities exist to leverage creative strategic partnerships that draw in stakeholders – including those from the private sector, non-profit organizations, foundations and philanthropic organizations, academia/University Transportation Centers (UTC), Federal agencies, and other public agencies – to advance smart city solutions. Successful implementation of a Smart City will rely on strategic partnering opportunities between public agencies and the private sector – especially for cities that have limited resources to bring to bear on the challenges they face.

Applicable Standards

• Standards generally (especially data dictionary standards), provide a common language for connecting diverse disciplines, technologies, and institutions.

Suitable Projects

• Public-private partnerships
Vision Element #8: Smart Grid, Roadway Electrification, and Electric Vehicles.

Description
This vision element includes strategies to leverage the smart grid – a programmable and efficient energy transmission and distribution system – in an effort to support the adoption or expansion of roadway electrification, robust electric vehicle charging infrastructure, and the acceleration of electric vehicle deployment. Opportunities exist for the integration of intelligent transportation systems with the smart grid and other energy distribution and charging systems. Electric vehicles are increasingly available across vehicle class (e.g., transit buses and medium duty vehicles) and price points. Smart-grid technology can enable infrastructure load (for example, from roadway lighting) to be shifted to off-peak periods, thereby flattening the daily load curve and significantly reducing both generation and network investment needs.

Applicable Standards
- NTCIP – Electrical and Lighting Management Systems
- Smart Grid Open Automated Demand Response (OpenADR)
- Electric Vehicle Open Charge Point Protocol (OCPP)

Suitable Projects
- Electric Vehicle Charging Networks
- Adaptive Roadway Lighting
- Electric Vehicle Fleets (buses, trains, government vehicles)

Vision Element #9: Connected, Involved Citizens.

Description
Connected citizens generate, share, and use data and information in new and useful ways. This vision element consists of strategies, local campaigns, and processes to proactively engage and inform citizens at the individual level by deploying hardware, software, and open data platforms in an effort to increase personal mobility.

One example of connected, involved citizens is leveraging the use of crowdsourcing. Crowdsourced data provides communication conduits through mobile technologies to connect citizens with city operators about a myriad of topics. In a successful Smart City, citizens would provide user-generated content to cities, opting-in to provide data from smartphones. Another example of connected, involved citizens includes leveraging broad access to open government data providing a platform for citizens and entrepreneurs to serve as co-creators and co-producers of new and innovative transportation services.

Suitable Projects
- Crowdsourced data
- Open data for 3rd Party Applications

Applicable Standards
- Open standards are essential to enabling 3rd party application development

Vision Element #10: Architecture and Standards.

Description
This vision element emphasizes complete and well-documented systems architectures and standards to support interoperability between systems. Because vehicles and travelers move broadly across regions, uniform operation that is accessible to everyone is essential for safe and efficient transportation operations.

Suitable Projects
• Projects to develop National (for services that are uniform across regions) and Regional ITS Architectures (for services that are unique to local regions)

Applicable Standards
• A number of standards organizations are mentioned in this paper.

Vision Element #11: Low-Cost, Efficient, Secure, and Resilient Information and Communications Technology (ICT).

Description
This vision element includes strategies and practices that advance information and communications technology (ICT) that is affordable, adaptable, scalable, efficient, secure and resilient. This may include telecommunications platforms, enterprise software, storage, visualization systems, and operations to inform decision making. ICT in a Smart City needs to be resilient, secure and respectful of privacy. Resilient design includes supporting standards, common technology architectures, and integrative policies. If one part of the system fails or is compromised, the entire system should not collapse, and the gap in service should be bridged effectively and restored quickly.

Suitable Projects
• Transportation and traveler communications networks projects

Applicable Standards
• Security and privacy standards
• System reliability and uptime standards

Vision Element #12: Smart Land Use.

Description
This vision element includes strategies and practices that ensure land use is optimized through a combination of planning and innovative deployments designed for a connected community that expands the range of transportation choices and access to employment, housing, education, and health services. A successful Smart City ensures that land use is efficiently optimized. Urban land use concentrates growth in compact walkable urban centers to avoid sprawl. It also advocates compact, transit-oriented, shared-use, walkable, bicycle-friendly land use, including neighborhood schools, complete streets (effective for all users), and mixed-use development with a range of housing choices. Smart land use values long-range, regional considerations of sustainability and citizen needs with the goals of achieving a unique sense of community and place; expanding the range of transportation, employment, and housing choices; equitably distributing the costs and benefits of development; preserving and enhancing natural and cultural resources; and promoting public health.

Suitable Projects
• Requires further investigation.

Applicable Standards
• Requires further investigation.

CONCLUSIONS

Smart Cities benefit from architecture and standards to enable deployment of interoperable technologies. Mature standards are available to support:
• Internet of Things
• ITS Equipment communications
• Connected Vehicles
• Public Transport
Additional research is needed to help support integration of automated (self-driving) vehicles, commercial vehicle logistics and information, land use planning, and public-private partnerships.

REFERENCES

Vision element descriptions were sourced from the following:

“Notice of Funding Opportunity Number DTFH6116RA00002, Beyond Traffic: The Smart City Challenge – Phase 2”, U.S. Department of Transportation, March 25, 2016

RESOURCES


For information about USDOT’s ITS Standards Program, visit: https://www.standards.its.dot.gov/

For information about the European Intelligent Transport Systems (ITS) Framework Architecture, visit: http://frame-online.eu/

For information on TPEG1 and TPEG2, standardized within the International Organization for Standardization as ISO/TS 18234 (TPEG1) and ISO/TS 21219 (TPEG2), visit http://iso.org

For information about the General Transit Feed Specification, visit https://developers.google.com/transit/gtfs/
Driving the Future Today – Global Status, Challenges and Opportunities for Cities from Connected and Autonomous Vehicles

Jonathan Spear, Director, Strategic Transport, Atkins Acuity (Singapore)
Ed Forrester, Intelligent Mobility Engineer, Atkins (Dubai, UAE)
International Road Federation, Middle East Regional Congress, October 2017

1. INTRODUCTION

Road vehicle applications linked to Connected and Autonomous Vehicles (CAVs) are developing rapidly in many parts of the World. Conditional Automation, with examples such as assisted parking and expressway autopilot, is expected to become commercially available within the next two to three years, whilst fully driverless road vehicle operation may be feasible and proven from a safety perspective from the mid-2020s onwards. In the Gulf, several Governments, including the Emirate of Dubai, have already set targets for driverless trip-making and the next few years are expected to see rapid advances not only in technology, but also the enabling environment of policy, regulation, social attitudes and sustainable business models. Whilst cities are likely to have different visions and levels of ambition to change, CAV technology may also combine with electric powertrains and on-demand mobility services to transform the road user proposition and experience.

This Paper sets out current trends in CAV technology and some key influences on the future road map, from the current focus on testing and proof of concept, through early adoption and progress towards market maturity. The key challenges for public decision makers and private industry are explored, including the current regulation of technology trials on public roads and medium-term changes to driver licensing, rules of the road and traffic law. The Paper also briefly considers some of the long-term implications of CAV adoption, on the provision of road infrastructure, as well as implications for urban planning, development control and city management.

2. THE CONTEXT OF INTELLIGENT MOBILITY

With populations and economies growing in cities across the World, urban transport networks will need to be better connected and integrated than ever before. They also need to utilise finite funding, land and other resources prudently and combine users, operators, service providers and regulators within a coherent and inter-linked “ecosystem.” With digital technology advancing, increasingly connected and populated via by Big Data, a wider range of transport solutions can, and will need to, be deployed which can achieve greater operational and environmental impact, and meet public expectations, with fewer resources.

Many current urban transport challenges stem from the inefficiencies resulting from more than a century of mass adoption of the private car. At the same time, conventional public transport systems have frequently been unable to offer a competitive alternative in terms of journey time, service quality and flexibility to user needs. Exploiting recent innovation in technology to respond to and overcome these limitations, concepts and products are rapidly developing around Intelligent Mobility which is emerging as the long-term and disruptive “future of transport.” Applications now rapidly evolving include:

- **Connected and Autonomous Vehicles (CAVs)**, otherwise known as Self-Driving Transport (SDT) and a host of other acronyms whereby vehicles will be capable of sensing data from their surrounding environment, infrastructure and the Internet, analysing and using it to control route, motion, trajectory, or provide a range of real-time information and other services, without the need for a human driver;

- **E-Mobility**, whereby automotive vehicles, such as cars and buses, and a range and Personal Mobility Devices, are powered by electric motors rather than the Internal Combustion Engine;

- **Mobility as a Service (MaaS)**, which presents new shared use models for buying and paying for travel enabled through technology, smart applications and pricing and drawing a new distinction between suppliers and operators of infrastructure and equipment, and brokers of services; and

- **Journey Planning and Management**, which provides integrated multi-modal travel information to users via a range of media, allowing them to make smarter travel choices on the move, whilst also enabling data for infrastructure and service operators so that decisions can be made on capacity management on a demand-responsive, flexible and commercial sustainable basis.
These measures, and the data platforms and systems underpinning them, are combining to offer potentially exciting prospects for enhanced and optimised urban mobility, operational performance, user experience, lower environmental impact and new commercial ventures. Moreover, much of the progress being made is being driven not by governments or public agencies, but by the private sector, which is itself subject to creative disruption, new business models and start-ups seeking to challenge established incumbents within the transport sector. Increasingly, it is clear that the mobility challenges facing cities in the 21\textsuperscript{st} century cannot be tackled with outdated planning, design and management approaches from the 20\textsuperscript{th}, and technology, systems thinking and data offers the opportunity for a new paradigm.

Intelligent Mobility, and the computing power, communications and data which support it, will enable more informed, multi-modal, personalised and flexible decisions to be made by network owners, service operators and providers and travellers themselves. In time, this will influence operator and end user needs and support sustainable economic growth and competitive advantage through knowledge creation and exploitation. However, this will only happen if policy makers and regulators within the public sector are clear about the objectives to be achieved, act proportionately and in a phased manner in balancing unconstrained innovation with public safety and risk management, and support the early market ahead of proven commercial viability, bankability, supply chain and mass adoption which may be demonstrated and taken forward over the medium- to long-term.

In this future vision, mobile devices and user accounts may connect to summon driverless cars, taxis and buses, or dispatch first and last-mile personal shuttles and pods to mass transit. With many cities testing, inviting and actively sponsoring or investing in technology research and development, it may eventually be possible to summon and pay for an automated ride with just one click on an app. Supporting this enlarged ecosystem within a shared economy stands to support a decisive shift away from private car ownership, with a substantial reduction in vehicle fleet size, an automated ride with just one click on an app. Supporting this enlarged ecosystem within a shared economy stands to support a decisive shift away from private car ownership, with a substantial reduction in vehicle fleet size, as well as traffic volumes, highway and parking capacity, required to serve travel demand for any given city.

Within Intelligent Mobility, the progressive introduction of CAVs is potentially a particularly transformational prospect. In essence, the concept relates to a vehicle which can drive itself between two points with substantially reduced or no manual input from a human driver. This is an evolution from simple autonomous features, such as anti-lock braking and cruise control which can be found in many existing commercially-available vehicles. Such features are useful to drivers, and improve safety and user experience, but fall short of a fully automated situation where a human driver is not needed to be in control for some or all of a journey, or potentially legally responsible for the behaviour of the vehicle. In this context, CAVs go further than current technology by automating four basic driver functions.

- **Navigation**: ability to plan and maintain a route;
- **Situational analysis**: ability to monitor the environment, aware of intervening objects and their movements;
- **Motion planning**: ability to maintain a course whilst avoiding collision with static or dynamic objects (including other vehicles and people); and
- **Trajectory control**: ability to maintain a stable speed and direction, including steering and acceleration.

To fulfil these functions, CAVs use a combination of internal digital maps, cameras, radar systems, sensors (Figure 1) and other information from satellites (such as GPS) to scan and determine their surroundings. They then use a central processor to determine the safest, quickest or otherwise most efficient and effective path to get to their destination. The central processor must be programmed with the “rules of the road,” both formal and informal for the country in question, so as to take decisions in a safe, predictable and acceptable manner. Linked to a computerised control system, mechatronic units and actuators then allow the car to accelerate, brake, steer and take other actions as needed.

Some of the functions above may be further enhanced by devices within the vehicle which can connect and communicate with devices, information and services within the infrastructure and external environment – so-called Connected Vehicles. Signals and commands can be received or sent from and to these devices, further allowing vehicle functionality to be enhanced in scope, speed and reliability. Forms of CV include Vehicle to Vehicle (V2V), Vehicle to Infrastructure (V2I), and Vehicle to Everything (V2X). Connected and Autonomous Vehicles (CAVs) leverage the advantages of both types of technology, automation and connectivity, to deliver the operationally best outcomes.

Much of the technology to enable fully connected and autonomous capability already exists, and many in industry argue functionality does not need significantly more hardware beyond that now being tested in many countries. However, more development is expected to take place around processing and software in terms of sophistication of interpretation of surroundings, predicting appropriate responses and instructing mechanical systems to perform the requisite functions. It is this area which is expected to see more significant development over coming years, proven by extensive real-world testing to ensure system reliability, resilience and predictability in multiple situations and circumstances, and the achievement of safety cases sufficient to allow regulatory approval and commercial deployment.
Beyond the technology, however, CAVs will need to develop in terms of several practical considerations, including regulation and standards, public perception and acceptance, ethics and liability, as well as implications for road infrastructure sizing, design and management. Solutions to these considerations must be achieved, beyond narrow development of the technology itself, if CAVs are to move from the current range of prototypes described below to early adoption, commercial application and mainstreaming within the mass market. Notwithstanding, some countries, such as the UK, Singapore and the USA, as well as Dubai in the GCC, have already started to address some of these issues as part of, or alongside, trials of self-driving vehicles on public roads.

It is important to note, at this juncture, that there is no single definition – or level – of vehicle automation. Instead, vehicles adopt different stages whereby increasing functionality and performance of driving tasks are transferred, wholly or in part, from a human driver to assistance or automation via an autonomous driving system and associated software. The Society of Automotive Engineers (SAE) International defines five levels in its 2014 J3016 Standard.

SAE Level 1 (Driver Assistance/Function-Specific Automation) has been widely adopted in the automotive market over the past two decades, for example through the introduction of cruise control and parking assistance. Applications equivalent to Levels 2 and 3 (Partial and Conditional Automation) are expected to become commercially available as a premium in certain vehicles over the next few years where increasing elements of steering, acceleration and deceleration can be undertaken automatically subject to the human driver continuing to perform other tasks, or where whole driving stages, such as parking or motorway driving can be safely and efficiently delegated.

The pathway to SAE Levels 4 to 5 – highly or fully driverless vehicle operation – is subject to more uncertainty. Through these levels, all driving tasks may become automated to the eventual end-point where the human driver can be “taken out of the loop” altogether. Indeed, vehicles can make trips and perform functions without the need for a driver to be present at all, and the need to include a steering wheel, brakes and other manual controls is, in principle and subject to industry technical standards and regulatory approval, rendered unnecessary.

Much driverless technology development to date has been within the automotive sector (Figure 2), progressed either by Original Equipment Manufacturers (OEMs) such as Nissan, BMW or Mercedes, or through research into autonomous driving systems from technology providers such as Waymo (formerly Google X), Mobileye, Delphi and, as recently confirmed by its CEO, Apple. However, this is not the only technology pathway, and progress is being made in personal and public transportation (e.g. Navya or Easymile), and also in freight and logistics, including the automation of road haulage and home deliveries, with Amazon, Ford, Otto and Starship Technologies all developing applications.

These pathways are not mutually exclusive and an extensive eco-system is becoming established spanning automotive, sensors and processors, software, mapping, operations and other support functions. Research from Vision Systems Intelligence lists a leading 125 out of an estimated more than 600 companies engaged in key elements of autonomous driving, ranging from large multi-national corporations to small tech and software start-ups. The value chain is expanding and evolving and is already seeing extensive merger and acquisition activity as key industry players look to consolidate the full set of technologies, competencies and support tools necessary to deliver integrated vehicle and service propositions to the market.

The pathways for road transport, of course, contrast with longer-standing trends in worldwide rail-based mass transit, where driverless systems, including those in Dubai, have been successfully operating for some years.
3. BENEFITS AND CHALLENGES OF CAV DEPLOYMENT

The key potential benefits and challenges of CAV adoption, based on a SWOT approach, are set out in Table 1.

When widely adopted, CAV technology has the potential to substantially reduce many of the existing negative externalities of personal road vehicle use inherent in modern society. These include road casualties, congestion, greenhouse gas emissions, air pollutions, noise, and business costs. According to many commentators, CAVs will also create benefits such as cashable cost savings for businesses and consumers, increased mobility, operational efficiency and enhanced and a more liveable urban environment. At high or universal rates of adoption, CAVs may potentially allow available road capacity to be used more efficiently, infrastructure sizing and footprint to be substantially reduced and facilities such as intersections, kerbsides, signage, street furniture, car parks and public transport interchanges re-engineered towards movement and utility for people rather than for the management motor vehicles.

These benefits may be achieved with enhanced confidence, or increased further in impact, when combined with other emerging technologies such as electric propulsion and Intelligent Mobility concepts around journey information, travel-brokering and value-added services direct to road users. Of particular significance are claims, propositioned above, that mass adoption of (electric) CAVs on a shared use model has the potential to transform the economics of personal car ownership, move away from a private model and substantially downsize the vehicle fleet size needed to serve cities. Some research, for example UITP, has put the latter reduction at up to 80%, citing evidence of recent declines in young people gaining driving licenses in cities such as London in favour of increased take-up of established public transport and the growing market in ride-sharing from companies such as Uber, Lyftt, Didi-Chuxing and Grab.

Such predictions are in line with prevailing orthodoxy around sustainable, integrated and inclusive urban transport which form the basis of many transport strategies and plans across the World. It is quite possible, however, that the benefits around this scenario may be overstated, and major discussion remains over whether adoption of CAVs will necessarily be paralleled with a decline in a predominant pattern of private ownership. It is also open to question whether a reduction in personal vehicle ownership, even if aligned with consumer tastes and achieved in practice, will necessarily translate into reduced vehicle kilometrage and traffic volumes on the road network. Whilst such an outcome is assumed by many, it is also possible that traffic volumes could increase if smaller shared vehicle fleets are used more intensively, there is substantial instances arise of empty running of CAVs to pick up passengers and goods, and the increased convenience of CAVs (including turning established notions of the value of travel time on their head) leads to additional induced travel demand from those who do not currently drive.

The questions of ownership model, behavioral response and induced travel in respect of CAV deployment are, in our view, major areas of uncertainty. Economic, social and market research is urgently required to understand from first principles the full extent of potential behavioral responses to CAV availability under different ownership and business

<table>
<thead>
<tr>
<th>Category</th>
<th>Application</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Automating Applications</td>
<td>Semi-Autonomous Electric Cars</td>
<td>Volvo’s V40, BMW i3, Nissan Leaf, Tesla Model S and X</td>
</tr>
<tr>
<td></td>
<td>Driverless Test Car</td>
<td>Google’s Fire, Mercedes F015 Luxury in Motion, Vauxhall’s Wildcat, Uber’s Volvo</td>
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<tr>
<td></td>
<td>Full Driverless Evolution</td>
<td>Mercedes’ Intelligent Drive, Audi’s Delphi, Nissan’s Autonomous Drive</td>
</tr>
<tr>
<td></td>
<td>Full Driverless Prototype</td>
<td>Waymo’s “Bubble” Car</td>
</tr>
<tr>
<td>2. Personalised and Public Transport Applications</td>
<td>Fully-Self-Driving Electric Ride-Taxi</td>
<td>Letz Pathfinder, Zapp’s, Ultra, RoboCar, Henk Future Mobility, GM En-V</td>
</tr>
<tr>
<td></td>
<td>Fully-Self-Driving Passenger Shuttle/Bus</td>
<td>Navya’s Arma, RoboCity’s EZ-10, Mercedes’ Future Bus</td>
</tr>
<tr>
<td>3. Autonomous Freight and Logistics Vehicules</td>
<td>Autonomous Warehouse/Building Operations</td>
<td>KARIS PRO System, RoboCoulier, Multishuttle Move, MOVESIX</td>
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<td></td>
<td>最后一级配送</td>
<td>Attenborough’s Harbour, Container Terminal</td>
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<td></td>
<td>Self-Driving Road Freight Vehicles</td>
<td>Mercedes’ Future Truck 2025, Volvo’s Gote Project, Otto/Uber ATG</td>
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<td></td>
<td>Last-Mile Delivery</td>
<td>Ford Autodelivery, Starship Technologies</td>
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<tr>
<td>4. Driverless Public Transport</td>
<td>Driverless Metro</td>
<td>Dubai Metro, Singapore’s NE Line, Vancouver Skytrain, VAL</td>
</tr>
<tr>
<td></td>
<td>Automated People Mover</td>
<td>Hong Kong International Airport, Dubai International Airport, Disneyland</td>
</tr>
</tbody>
</table>

Figure 2. Current Trends in Connected and Autonomous Vehicle Development
models, and the resulting impact on demand and use of available highway capacity. It may well be that in order to
deliver the reduction in traffic and congestion oft quoted by CAV proponents, regulation, pricing and other mechanisms
may be needed to manage demand for travel, including ensuring that ride sharing and public transport services are
managed in an integrated manner rather than competitively.

Table 1. Strengths, Weaknesses, Opportunities and Threats to CAV Adoption on Urban Transport

<table>
<thead>
<tr>
<th>Strengths</th>
<th>Weaknesses</th>
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<tbody>
<tr>
<td>Bundling of CAV technology with other elements of Intelligent Mobility</td>
<td>Technology not yet mature to safety case</td>
</tr>
<tr>
<td>Road safety and reduction in casualties</td>
<td>Most traffic law/rules of road based on human driver</td>
</tr>
<tr>
<td>Efficient and resilient asset use</td>
<td>No agreed technical standards</td>
</tr>
<tr>
<td>Reduced infrastructure sizing and footprint</td>
<td>Current low speed and performance</td>
</tr>
<tr>
<td>Smart city brand and reputation</td>
<td>Limited human factors research e.g. No clarity on handover between human driver &amp; AV system</td>
</tr>
<tr>
<td>Reduction in vehicle ownership and kilometres</td>
<td>Higher complexity and costs of maintenance</td>
</tr>
<tr>
<td>Reduction in, and management of, congestion</td>
<td>Higher capital and operating costs</td>
</tr>
<tr>
<td>Reliability and better journey times</td>
<td>Public perception currently unclear/sceptical</td>
</tr>
<tr>
<td>E-mobility and decarbonisation, leading to energy and fuel efficiency</td>
<td>Competitive threats to conventional Public Transport</td>
</tr>
<tr>
<td>Social inclusion for those currently unable to drive</td>
<td>Substantial benefits only through high market take-up and occupancy/shared use</td>
</tr>
<tr>
<td>User travel experience, including reduced stress and more productive use of travel time</td>
<td>Interaction between CAVs and human drivers may be unsafe, or require conservative programming assumptions</td>
</tr>
<tr>
<td>Business cost savings and prices for consumers</td>
<td>Uncertain transition period and processes</td>
</tr>
<tr>
<td>Reducing investment and public funding burden on Governments and public agencies</td>
<td>Supply chain evolving &amp; immature for key components, although substantial M&amp;A activity to consolidate</td>
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<table>
<thead>
<tr>
<th>Opportunities</th>
<th>Threats</th>
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<tbody>
<tr>
<td>Potential for CAVs to dominate urban mobility and integrate/span market from private to public transport</td>
<td>New safety risks (e.g. system failure, software errors)</td>
</tr>
<tr>
<td>Combination with Mobility as a Service to maximise reduction in private vehicle ownership</td>
<td>Cybersecurity</td>
</tr>
<tr>
<td>Reducing driver training and licenses amongst younger generation in some advanced cities</td>
<td>Lack of integration/inter-operability of standards and regulations across federal and national borders</td>
</tr>
<tr>
<td>Diversification of current public transport operators and business strategies</td>
<td>Loss of driving job/businesses (with political resistance)</td>
</tr>
<tr>
<td>New freight, logistics and supply chain models</td>
<td>Uncertainty of life cycle costs, competition and new revenue streams undermining investment confidence</td>
</tr>
<tr>
<td>New business models for transport infrastructure, assets and services</td>
<td>Loss of revenue (e.g. Parking, Road Tolls)</td>
</tr>
<tr>
<td>Scope for major initiatives to be led by private sector investment, coming from established transport operators, new entrants and investors seeking commercial propositions</td>
<td>Privatisation of urban transport (e.g. Uber) at expense of public authorities</td>
</tr>
<tr>
<td>New job creation in technology and customer-facing roles with upskilling of workforce</td>
<td>User scepticism and resistance</td>
</tr>
<tr>
<td>Regaining/recycling urban space through reduced road and parking needs, supporting enhanced public realm and liveable cities</td>
<td>Behavioral response may induce demand, leading to traffic generation and congestion, unless carefully managed</td>
</tr>
<tr>
<td></td>
<td>Urban sprawl as a resulting of extended trip distances</td>
</tr>
<tr>
<td></td>
<td>Interaction between CAVs and human drivers</td>
</tr>
<tr>
<td></td>
<td>Uncertain transition period and processes</td>
</tr>
<tr>
<td></td>
<td>Lack of clear public regulatory approach and/or over-regulation stifling innovation, not assisted by lack of digital technology skills within Government &amp; public sector</td>
</tr>
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</table>

The road safety benefits resulting from mass take-up of CAVs, likewise, are potentially compelling. With over 1.2
million road deaths worldwide every year, and between 75% and 90% claimed due to human error, a concerted effort to
take human beings out of the driving loop could save hundreds of thousands of lives, untold suffering, traffic disruption
and economic cost. In low income economies, where casualty rates are often higher, such benefits could be an order of
magnitude greater than those pertaining to decongestion and journey time savings. Against this, new forms of safety risk will undoubtedly emerge, including failures due to poor maintenance, software errors and cybersecurity threats. Safeguarding against these eventualities will carry substantial implications for operating costs, as well as shifts in liability, certification and insurance from personal negligence to product warranty and consumer law.

Others have pointed to the transition period in the CAV road map from early adoption toward market maturity which will see of human drivers and autonomous vehicles inter-acting at various proportions on the road network. The latter raises the potential for an increase, rather than a reduction, in accidents in the short-term, requirements for physical segregation or, if mixed traffic, the need to programme CAVs to drive defensively matched to match human behaviour. Similarly, research from Atkins has shown that substantial reduction in journey delay of up to 40% may be possible on urban roads, but only when CAV penetration reaches 50 – 75% of vehicles in the fleet. Human factors research remains a priority, including not only the question of whether to segregate CAVs, but the question of a “safe” handover process from an autonomous driving system to a human driver and whether such a stage is even desirable.

Assuming some of the ownership, behavioral response and transition issues cited above can be adequately addressed, it does appear that there are potentially major benefits from mass CAV adoption for urban planning and the future development and management of cities. Under current planning regimes, many cities have on average 30% of their surface area devoted to roads and associated facilities such as parking. The design and layout of the streetscape is also heavily influenced by the need to direct and manage motor vehicles. With an average car spending 95% of its time empty and parked, and only 5% occupied and operating on the roads, this is clearly a hugely inefficient use of space, both operationally and economically. In principle, CAVs may have the potential to influence urban form through more efficient road use, less parking and reduction in infrastructure size, and a simplification of street layout. By some estimates (e.g. Spear, 2016), this could ultimately release 15 to 20% of developable area for alternative uses, for example, enhanced greenery and public realm, recreation and leisure facilities, or new development, either on a fully commercial basis or for affordable housing, business premises or community facilities.

Figure 3. Concept of Future Town Centre with Self-Driving Vehicles (Singapore Ministry of Transport)

In Singapore, the Government is committed to a “car-lite” vision which not only envisages the progressive automation of personal on-demand and fixed public transport services, but parallels this with removing major traffic flows from local neighborhoods, investing in networks and facilities for pedestrians and cyclists and bringing forward affordable housing designed around people rather than vehicles. Such a vision becomes more economically viable if the footprint of road and parking infrastructure can be reduced and made more responsive to demand through data, pricing and personalised journey planning. Assuming the desire reductions in car ownership can be achieved, a key challenge will be the development of revised highway design and traffic management guidelines, based on empirical research, which give space back to communities whilst maintaining vehicular circulation and access.

It is clear, therefore, that robust evidence of the extent and timing of benefits of CAV adoption, and the implications for public policy, are still largely hypothetical, under study and potentially overly-simplified. Equally, there is a need to better consider dis-benefits, some of which can be conceptualised now (e.g. concerns over privacy, higher maintenance complexity and costs), and others which are largely unknown and may emerge in due course. For example, an indirect consequence of a sharp reduction in road fatalities resulting from CAV adoption may be a fall in the number of organ donations available for transplant within hospitals. This may require greater investment in stem-cell and other medical research to meet demand and a degree of pre-planning by national health agencies.
4. FUTURE ROAD MAP AND KEY INFLUENCERS ON THE PACE OF CAV DEPLOYMENT

As well as uncertainties over the precise nature and balance of benefits and challenges of CAV take-up, a key area of debate is the timescale to be applied to development, early adoption and mass market deployment of the technology.

Figure 4, adapted from KPMG (2015), presents a potential road map for the adoption of CAV technology at the different SAE levels towards 2030. Under this timeline, Level 3 applications such as intersection assistant, highway autopilot or remote parking could become available towards the end of this decade, with highly and fully autonomous technology become commercially available between 2025 and 2030. Considerable hurdles remain to be overcome, such as refinement of prediction and decision-making algorithms, human factors and cyber security. Level 3 will assist in this, support progressive regulation, and eventually bring the technology to a state of proven maturity.

Figure 4. Potential Future Road Map for CAV Adoption

However, while literature on the topic is generally consistent in forecasting the technical feasibility of Full Automation, it is more variable on the development of fully mature commercial applications at acceptable cost, social acceptance and consistent certification by regulators. A reasonable assumption is that CAV technology will not evolve to the level of market penetration required to enable significant changes in public road infrastructure standards, traffic management and the transformation of driving laws and regulations until well after 2030.

Ultimately, CAV adoption will be governed by three inter-related factors (Figure 4) of technological development, laws and regulation, and social acceptance by the public. The latter includes not only the perception that CAVs are safe, but an appreciation of the user benefits and a willingness to pay or adopt a new ownership or use model to realise those benefits. The inter-play of these factors will determine the aggressiveness or conservatism of the pace of change.

To an extent, the pace of CAV adoption can be predicted based on previous automotive technologies, such as automatic transmission and airbags (Litman, 2017). Based on this, most new technologies require decades of technical development and growth to saturate their potential markets, and in many cases, may never become universal. In the automotive sector, airbags have had the shortest cycle and the most complete market share, due to technical standards now in place in many countries. Automatic transmission, by contrast, required more than five decades for prices to decline and quality to improve, and the technology is still not universal. Hybrid vehicles are still developing after 15 years on the market, and are only now showing signs of significant market break-through, although may yet be overtaken by Battery Electric Vehicles. This suggests that new vehicle technologies generally may require two to five decades from commercial availability to market saturation, and without government mandates, may not be universal.

If CAV deployment follows the patterns of other vehicle technologies it will take one to three decades to dominate vehicle sales, plus one or two more decades to dominate vehicle travel as the fleet turns over. Even at market saturation it is possible that a proportion of vehicles and travel may continue to be self-driven, unless mandated otherwise by regulators. This points to the 2050s and beyond for most vehicles in the market to be capable of self-driving, although as indicated below, changes in the ownership structure and business model may accelerate the transition.
Figure 5. Differing Influencers on Speed of CAV Adoption

There is some evidence that some sub-sectors could grow more rapidly as appreciation of the long-term potential grows and the technology combines with new business and service models. Within the Intelligent Mobility sphere, examples such as Uber, Lyft, Moovit and Waze have come from nowhere to become major players in a very short timescale; investment in the market in ride-hailing alone has grown rapidly from less than US$ 200 million in 2013 to over US$ 11 billion in 2015 and topping US$21 billion by mid-2016. Waymo (formerly Google X) is collaborating with Fiat Chrysler and Lyft on the development of self-driving technology and BMW, Baidu and General Motors have opened sizeable Silicon Valley offices and dedicated hundreds of millions of dollars to their own product development and to acquire autonomous vehicle start-ups.

Indeed, some commentators have spoken of the “Convergence of Things” as different IM concepts begin to coalesce within a wider ecosystem in terms of hardware, physical systems, software and data. It is no coincidence that the CEO of Apple recently raised the prospect of a combination of three vectors of change around self-driving technology, electric power trains and ride-hailing to produce a new and compelling business model. On that basis, and provided regulators so permit, there is, in the next few years, the prospect of an “iTunes” moment which moves technological experimentation to a new integrated hardware and service proposition which successfully – and more rapidly than previous automotive technologies – achieves a transformation in public profile, appeal and demand.

In parallel, established transport operators, for example of city bus and taxi networks, especially those with a global business and footprint, may also have a role to play in accelerating deployment if they regard CAVs, rather than a new competitive threat, as a diversification of their brand and operating model. This is the case, for example, with Singaporean transport operator SMRT investing in 2getthere, the Dutch Automated Transit firm; or Transdev launching major first-last mile connection initiatives in North America with EasyMile. This mimics automotive manufacturers acquisition of numerous mapping, sensor and processing firms to deliver integrated vehicle and service solutions. Further proactive development of the sector looks likely to bring capital investment, merger and acquisition of key technology suppliers, commercial acumen and public-facing experience to what is still a relatively immature sector, with large public and shared transport operators, and automotive manufacturers themselves, rather than the public, potentially in line of early adoption at city level.

5. SOME PRELIMINARY CONCLUSIONS

The discussion above draws us several preliminary conclusions on the current state of play with CAV development:

- CAV technology is already commercially available at close to Level 3 Conditional Autonomy, or about to become so in the next few years, in terms of being deployed on public roads and this will impact on, and require a response by, Governments, city authorities and the public;

- The key technological challenges towards Full Autonomy will be solved and resilient within the next 5 – 10 years, but this does not equate to consumer acceptance, affordability and market take-up, which will likely take considerably longer, depending on business model;
CAVs have multiple anticipated benefits, many of which address challenges and would deliver improvements for cities, including potentially far-reaching impacts on traffic flow, congestion, infrastructure sizing, urban design and public realm. In many ways, CAVs will be as much an urban planning, as a transport, agenda; There is increasing literature on CAV impacts, speed and shape of implementation, but there remains much speculation, uncertainty, claim and counter-claim, and empirical evidence is urgently needed, especially in areas such as social attitudes, human factors, demand drivers and behavioral change, which could determine the balance of positive and negative outcomes. Table 2 below defines some of the research requirements; Full automation is some years away in terms of being commercially available at affordable cost, but does fall within medium- to long-term time horizon of many city transport plans. Therefore, transport planners should be mapping out scenarios, strategies and road maps now and remaining flexible to the pace of change; and Long-term CAV development will require strong collaboration across stakeholders within government, regulators and industry, and it will be essential to rethink partnerships, and how early relationships can be consolidated, to clarify roles, manage respective expectations and produce mutually beneficial outcomes.

Finally, we note that CAVs are currently mainly an agenda for advanced economies. There is limited focus on introduction or deployment of the technology for developing countries. We believe this is a major gap. Whilst there are undoubted technical challenges, skills and capacity issues, in adopting CAV approaches in many emerging economies, securing some basic applications and supporting their incorporation into vehicle standards and driving practices could secure major and widespread benefits, not least in the area of road safety, with low income communities often suffering disproportionately from deaths and serious injuries from road accidents. Bodies such as World Bank, Asian and African Development Banks, and national development agencies, need to be more proactive in this area.

Table 2. Key Areas for Further Research in CAV Development and Deployment

<table>
<thead>
<tr>
<th>Technology</th>
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<tbody>
<tr>
<td>• Component/Systems/Integration Development, Prototype and Commercialisation</td>
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<tr>
<td>• Machine Learning and Artificial Intelligence</td>
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<tr>
<td>• Technical Standards and Safety Assurance</td>
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<tr>
<td>• Data Protocols and Governance</td>
</tr>
<tr>
<td>• Human Factors, Key Interfaces and CAV Operational Parameters</td>
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<tr>
<th>Planning</th>
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<tbody>
<tr>
<td>• Car Ownership, Car Sharing and Ride Sharing Models – Future Trends in Access vs Ownership</td>
</tr>
<tr>
<td>• Social Attitudes to Take-Up, User Needs and Influence on Travel Attitudes and Behaviour</td>
</tr>
<tr>
<td>• Network Modelling and Traffic Simulation to Understand Impacts (at City and Local Levels)</td>
</tr>
<tr>
<td>• Adoption Curve and Conversion Rate to Achieve Impacts &amp; Justify Infrastructure Changes</td>
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<tr>
<th>Policy</th>
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<tbody>
<tr>
<td>• Overall Policy, Strategy, Programme and Timeline</td>
</tr>
<tr>
<td>• Regulatory Approach and Progressive Review and Reform of Regulations</td>
</tr>
<tr>
<td>• Transition Period – Length, End-State and Interim Management Strategies for Mixed Traffic</td>
</tr>
<tr>
<td>• Governance, Business Models, Private Sector Involvement, Liability and Insurance</td>
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<table>
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<tr>
<th>Engineering and Design</th>
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</thead>
<tbody>
<tr>
<td>• Highway Capacity Allocation, Design, Engineering, Operations and Asset Management</td>
</tr>
<tr>
<td>• Parking, Charging, Communications and Other Facilities</td>
</tr>
<tr>
<td>• Urban Planning and Design</td>
</tr>
<tr>
<td>• Public Realm Management</td>
</tr>
<tr>
<td>• Implications for Traffic Impact and Development Control</td>
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<tr>
<th>Risks</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Cyber-Security and System Integrity</td>
</tr>
<tr>
<td>• Burden of Regulation - Balance of Promoting Innovation and Deployment vs. Public Safety</td>
</tr>
<tr>
<td>• Managing Disruption on Driving-Related Jobs and Sectors and Unintended Consequences</td>
</tr>
</tbody>
</table>
6. THE CURRENT FOCUS ON TECHNOLOGY TESTING AND VALIDATION

Ahead of the commercialisation of Level 3 automation and further development towards Levels 4 and 5, the key current stage of CAV-related activity is on technology trials, testing and demonstration projects to show and validate, within suitable environments, early concepts, systems and operational practices, as well as inform a range of policy, regulatory and practical challenges.

Such trials are underway and at notable degrees of progress in the UK, Sweden, Germany, USA, China, Japan, Singapore and Australia. In the West of England, for example, the VENTURER Project includes 13 public, private and academic organisations cooperating to simulate and test CAV technology in a real-world environment, study key operational, economic and commercial issues and understand the blockers and drivers to wide scale CAV adoption. The three-year project is co-funded by the UK Government and the partners themselves, alongside equivalent trials in the Midlands and London, aiming to position the UK at the forefront of CAV technology and associated economic benefits from the value chain. Similarly, Singapore has launched the Singapore Automated Vehicle Initiative (SVI) which includes technology testing at one-north, a business park precinct and the forthcoming opening of a new off-road test track. In the UAE, Dubai Roads and Transport Authority is planning a Self-Driving Transport Competition; this will showcase CAV technologies across several use cases and environments and raise public awareness as part of its programme for 25% of all trips within the Emirate to be driverless by 2030. All these exercises cover automotive applications for passengers or freight, as well as personal or small group pods or shuttles designed for pedestrian or semi-controlled environments, and first and last mile connections to public transport.

Several countries, including UK and Australia have developed codes of practice, or equivalent frameworks, to guide the conduct of such trials within the prevailing legal and regulatory environment, balancing the need to promote innovation with the protection of public safety. In both instances, these codes are the first step in laying the basis for longer-term regulatory reform and change to allow driverless road transport, for example by examining driver licensing arrangements, issues of accident liability, traffic law and the long-term adaptation of vehicle and highway standards.

To plan resources, manage risks and produce clear technical outcomes, current testing efforts might focus usefully on CAV applications for the movement of people and goods which are likely to be early adopters. These might include, for example, first and last mile connections to public transport as indicated above, local circuits within business or retail parks, remote car parking or support for mobility impaired people within a pedestrianised environment. Certain closed environments, such as airport airside road networks, have also been suggested as offering a good test and early deployment opportunity, especially given the stringent operational control and safety-focused regulations and culture which governs civil aviation. Ideally, applications should be sufficiently different in terms of use cases, but provide overlap in terms of the fundamentals relating to technology choices, operating models and journey management.

Many current trials appear to have adopted a two to three-year trial period, allowing adequate time for planning and preparation, actual vehicle testing at various stages of complexity, and adequate collection and analysis of data on performance and user perceptions. Shorter periods may be, and are being, considered, but still need to be sufficiently robust to demonstrate a true proof of concept under all real-life operational circumstances and conditions.

Ideally, testing should take the form of “cycles of increasing complexity” where scenarios are developed from a relatively basic level of simulation in the lab, through simple test runs without other vehicles or people being present up to full operational “real world” representation at various degrees of challenge. A proof of concept also needs to be tightly framed to have defined success criteria, objectives and measurable outcomes. The trial must ultimately demonstrate whether the desired outcomes are achievable and the technical and practical pre-conditions to deliver them. In order to structure a detailed task breakdown needed for such demonstration, three stages are relevant covering pre-trial considerations, aspects during testing and the validation and review of results. These stages should also link to an overall “vision” and medium-term strategy which guides the deployment of CAVs within whichever urban or other operational environment is defined. Table 3 defines the key stages.

The scope and length of testing needs to consider local context and circumstances, for example, considering existing vehicle and driver work patterns, volumes and elements to be carried, infrastructure and operational constraints, and governance/managerial arrangements. These will help build up a statistical reference point for validation and benchmarking of performance.

It may also be necessary to review current vehicle, driving regulations and insurance regulations, ensuring that testing complies with these or new temporary codes that may be issued and approved by regulators. As noted, some countries, and where relevant sub-national jurisdictions, have been proactive in preparing CAV testing codes of practice. Typically, these require the presence of a qualified test driver, irrespective of SAE Level, fail-safe handover procedures between automated and manual drive, comprehensive data collection, and risk assessment of all stages.
Table 3. Key Stages of CAV Testing

<table>
<thead>
<tr>
<th>Pre-Trail Considerations</th>
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<tbody>
<tr>
<td>• Breakdown of test requirements including objectives, quantification through KPIs and safety case</td>
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<tr>
<td>• High level scoping by use case, including vehicle and trip type, review and design of routes, nodes and areas over which the trial will be run and the facilities which will be accessed</td>
</tr>
<tr>
<td>• Identification of key CAV components, including hardware, equipment and systems, communications and software, and facilities such as control room for overall monitoring and management</td>
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<tr>
<td>• Technology and product scoping and procurement arrangements</td>
</tr>
<tr>
<td>• Review of infrastructure requirements, including adaptations (e.g. segregated lanes, intersection signaling) required specifically to ensure safety, user assurance and protection of third parties</td>
</tr>
<tr>
<td>• Relevant policy and regulatory issues, including regulatory approval to conduct the trial, conditions set to ensure safety, data collection, and management of impacts on other road users</td>
</tr>
<tr>
<td>• Skills and gap analysis and proposals for operational capacity, capability and resources to undertake the trial within sponsoring or organising entities</td>
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<tr>
<td>• Monitoring capability, including vehicle tracking, systems reporting and public surveys</td>
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<tr>
<th>In-Trial Considerations</th>
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<tr>
<td>• Detailed test scenario development, whereby key vehicle and journey functions, dimensions and operating performance are quantified</td>
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<tr>
<td>• System architecture development, together with sensor/processing definition, interfaces and integration with automotive controls</td>
</tr>
<tr>
<td>• Data capture, analytics and operational control dashboard creation</td>
</tr>
<tr>
<td>• Definition of trial stages based on working cycles of increasing complexity in terms of volumes, obstructions, interaction with other traffic, pedestrian movements, &amp; response to operational variation</td>
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<tr>
<th>Validation and Review</th>
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<tr>
<td>• Adjustments to CAV tech specification, characteristics and operating procedures in light of results</td>
</tr>
<tr>
<td>• Linking test outcomes to pathway, form and operating model for operational deployment</td>
</tr>
<tr>
<td>• Structuring of functional requirements aligned to operating model</td>
</tr>
<tr>
<td>• Integration requirements across systems</td>
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<tr>
<td>• Governance for data collection, storage and processing</td>
</tr>
<tr>
<td>• Policy and regulatory lessons and implications, including type approval if technology is demonstrated as safe and meeting operational objectives</td>
</tr>
<tr>
<td>• Collaborative arrangements for CAV development and operation, including regulatory and business model</td>
</tr>
<tr>
<td>• Technical review and identification of next steps for deployment</td>
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As noted, as well as operational safety and technical performance, an important aspect of testing is public reaction to the CAV technology being deployed. This is important to raise awareness and overcome the split between public support and skepticism to use CAVs which many surveys have shown to date. However, assessment of public attitudes is also important to shape the human interface aspects of future vehicle design, but also to determine future propensity for driverless trip-making and delivery model in terms of shared versus private use. Early survey evidence may also give greater insight into the likelihood that mass CAV adoption will result in better managed and more orderly traffic flows, or a high degree of induced demand which will exacerbate, rather than improve, congestion and network conditions. As noted, regulation, pricing and other forms of demand management may be required to guide and influence travel behaviour in this regard.
7. CONCLUSIONS – A POSSIBLE TIME LINE FOR CAV ADOPTION AT SOCIETAL LEVEL

Table 4 looks beyond the current focus on technology testing and the earlier discussions on the wider influencers on the CAV adoption, to set out a potential timeline for CAV adoption and impacts between now and 2060. The horizon is a generic and illustrative one for discussion, but does attempt to set out the potential developments and milestones which decision makers in government and industry may need to consider when in planning ahead. Some countries and cities may be ahead of this timeline, ambitious to drive technological change and the competitive advantage which goes with this, whilst others will be more reactive, regulating and adopting once global standards have been largely set.

Table 4. A Generic Timeline for CAV Technology Development and Deployment

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<tbody>
<tr>
<td>Evidence gathering &amp; assessment of benefits and challenges</td>
<td>First deployments of Level 4/5</td>
<td>Regulatory maturity</td>
<td>Mainstream across whole road network and all operations</td>
<td></td>
</tr>
<tr>
<td>Strategy &amp; planning</td>
<td>Clarity and established legal and regulatory practice</td>
<td>Integration with public transport</td>
<td>Full integration of CAVs, infrastructure, city control systems, urban design and public realm</td>
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<tr>
<td>Regulatory review and code of practice for public testing</td>
<td>Deployment of priority commercially viable products/services</td>
<td>Infrastructure design/O&amp;M</td>
<td>New urban transport governance models</td>
<td></td>
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<tr>
<td>Business and safety cases</td>
<td>Disruption of public transport, taxi and logistics</td>
<td>Changing attitudes, and increasing societal adoption</td>
<td>Fundamental vehicle transformation and design standards</td>
<td></td>
</tr>
<tr>
<td>Progressing test locations, use cases and trials</td>
<td>Technology bundling (EV, IM), linked to new governance &amp; business models</td>
<td>Revision of urban and public realm planning</td>
<td>Artificial intelligence</td>
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<tr>
<td>Early codes of practice (testing)</td>
<td>Maturity of supply chain</td>
<td>Transition management towards majority of fleet being CAVs</td>
<td>Possible restriction of manual driving at societal level</td>
<td></td>
</tr>
<tr>
<td>Planning and impact studies</td>
<td>Urban planning and design to future-proof CAV &amp; IM</td>
<td></td>
<td>Urban transformation, guided by transport demand management</td>
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<tr>
<td>Early international and regional technical standards</td>
<td>Early infrastructure design/O&amp;M</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Early governance &amp; business models</td>
<td>Early transition management recognising mixed use between CAVs and human drivers</td>
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</table>

To conclude, CAV technology is already with us, developing rapidly and has the potential to change road-based urban mobility in our lifetimes in a way not seen since the invention of the internal combustion engine in the nineteenth century. For many, it is no longer a question of “if,” but “when” the full extent of disruption and transformation will occur. The first iPhone, an early enabler of Intelligent Mobility, was introduced a mere decade ago and Apple itself has now publicly announced its plans to develop an autonomous driving system towards the end of the decade. Research by other OEMs and technology companies could likewise revolutionize the road user experience, with far reaching consequences for business models, highway design, operation and management, and the layout of urban areas.

Despite this, there are good reasons for some caution in predicting the future. There remains considerable uncertainty over the previse balance of CAV benefits, challenges and impact on personal travel behaviour. This is especially the case during the transition period, itself of uncertain duration, when only a proportion of road transport is autonomous and then only under certain conditions. Despite multiple generic claims of impact, the devil will be in the detail and in-depth research is urgently needed to have greater confidence in actual effects, positive and negative, combined with empirical revealed data as Level 3 Conditional Autonomation comes to market. Likewise, policy makers will have a major challenge to keep up with innovation and change, deciding how to progressively regulate testing, early adoption and mass deployment at various stages of technology readiness and maturity, and do so in the context of international, as well as national, industry standards.

However, this uncertainty should not be used as an excuse for indecision, prevarication and inaction to rise to the potential enabled by technological innovation, whilst directly addressing some of the difficult social, political and ethical issues. Public road and transport agencies across the World will need to build their planning approaches, skills and capacity to not only respond, but lead the way and, once the technical evidence is clear, firm and timely decisions must be made.
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Effects on Istanbul Traffic of the Eurasia Tunnel and Removal of Additional Lanes

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ABSTRACT

Eighteen percent of Turkey’s population lives in Istanbul, one of the largest and traffic congested cities in the world. The most significant investments that have been made in this area include projects connecting the European and Asian continents by rail and roads. In recent years, Istanbul Metropolitan Municipality (IMM) and the Turkish Ministry of Transport, Maritime Affairs and Communications have undertaken three major projects to alleviate traffic congestion in the city. The first, Marmaray, makes a significant contribution to public rail transportation between the two continents by way of a tunnel under the Bosphorus strait. The second, Yavuz Sultan Selim highway bridge, is used by cars and heavy trucks for passenger and goods transit. Finally, the Eurasia Tunnel allows passenger cars to drive between the continents under the Sea of Marmara. This paper investigates the impact of the recently opened Eurasia Tunnel on traffic conditions in Istanbul. The effects of discontinuing the use of extra lanes during peak hours on the existing two bridges is also considered. Intercontinental travel times before and after the opening of the Eurasia Tunnel, changes in average speeds and the volumes of vehicles passing over the bridges are also reported.

Keywords: Effects of Eurasia Tunnel, Additional Lane, Traffic, Bluetooth-Smart Sensor, IMM.

I INTRODUCTION

The daily increase in the population of Istanbul, combined with the boost in private vehicle ownership due to rising incomes, and the consequent growth in the number of new vehicles added daily to the city’s traffic. These factors give rise not only to an increment in the intensity of urban traffic, they also lead to insufficient access to the existing transportation network and to traffic queues. When the urban transport movements in Istanbul are examined, it demonstrates that a bottleneck effect is created especially during bridge crossings between two continents where the number of daily commutes are high.

Existing bridges that provide transit between two continents play an important role in regulation of mobility and urban traffic in Istanbul which flows on the east-west axis. In this paper, we examine the effects of the Eurasia Tunnel, which reduces the traffic load of existing bridges and contributes to a more balanced urban transport, and the parallel effects of removing the additional lanes that had previously been applied on the bridge crossings during peak hours. The scope of this examination covers the impact of the Eurasia Tunnel and the removal of additional traffic lanes on the D100 Highway and the TEM (E-80) Express way and on traffic using the July 15 Martyrs Bridge and Fatih Sultan Mehmet Bridge crossings. Technical information about the Eurasia Tunnel is presented first, followed by information including changes in average travel times and average speeds on D100 Kozyatağı and D100 Kozyatağı → Atatürk Airport routes, before and after the opening of the Eurasia Tunnel. Changes in the number of vehicles crossing bridges and travelling through the Eurasia Tunnel are
also addressed.

2 THE EURASIA TUNNEL PROJECT

The Eurasia Tunnel (Istanbul Strait Road Tube Crossing Project), which connects the continents of Europe and Asia, is constructed between Kazlıçeşme and Göztepe on a 14.6km route. It is a two-storey tunnel under the Bosphorus with a length of 5.4km. The Eurasia Tunnel is intended to contribute to relieving traffic problems on this congested route and reducing travel times between the two continents. In the Eurasia Tunnel Project, the two continents have been connected by a tunnel boring machine (TBM) over a 3.344-km section underneath the Bosphorus.  

Within the scope of this project, road expansions and improvement works were carried out over a route totaling 9.2km across the European and Asian continents. Access roads between Sarayburnu-Kazlıçeşme and Harem-Göztepe were expanded, and new intersections, underpasses and pedestrian overpasses were constructed. The Eurasia Tunnel, as seen in Figure 1, is the first two-storey highway tunnel in the world which is under the sea.

![Figure 1. Eurasia Tunnel](image)

2.1 Goals of the Project

There is a heavy traffic along the east-west axis in Istanbul, especially on the bridge crossing routes during peak hours. The Eurasia Tunnel aims not only to reduce the traffic loads on the July 15th Martyrs Bridge and Galata and Atatürk Bridges which are located above the Golden Horn, but also to distribute the traffic between the two continents more evenly and to contribute to more balanced urban transportation overall.

Average travel time between the start and end points of the project is about 42 minutes, rising to a range of 60–100 minutes when traffic density is high. This project, which caters for a daily capacity of 130,000 vehicles, aims to reduce this travel time to 15 minutes, and to minimize the emission of harmful gases into the atmosphere, including CO, PM, NOx, SO2 and exhaust gases caused by extended travel times in congested traffic. Reducing the number of vehicles entering the historic peninsula and decreasing the total number of transit trips within the peninsula are also among the project goals.

2.2 Project Route

The total length of the Eurasia Tunnel is 14.6km, comprising three main parts:

**Europe Section:** A U-turn was constructed on Kennedy Street to provide entrance to the underpass section from Kazlıçeşme to Sarayburnu, and a disabled-accessible pedestrian bridge was constructed as an overpass. The current road, which is about 5.4km in length, was widened to have 2x4 lanes from the 2x3 and 3x2 lanes that existed prior to the Eurasia Tunnel Project.

**Bosphorus Crossing:** The toll gates and operation center are located at the western entrance of the 5.4km long two-storey tunnel which passes under the sea floor of Bosphorus. Ventilation chimneys are situated at both ends of the tunnel. The tunnel has been constructed with two lanes on each level.
Anatolian Peninsula: Two interchanges were constructed on the D100 Highway towards Göztepe, with disabled-accessible pedestrian overpasses. The current road, which is about 3.8km long, was widened to have 2x4 and 2x5 lanes, as compared with 2x3 and 2x4 lanes before the Project.

Route of Eurasia Tunnel Project is seen in Figure 2.

3 METHODS USED IN IMPACT ANALYSIS

There have been noticeable improvements in Istanbul’s urban traffic, especially in intercontinental transits, following deployment of the Eurasia Tunnel Project. Average speed and travel time data obtained from bluetooth sensors were used to calculate and compare these improvements. These sensors have been installed on the inner-city road network of Istanbul by Department of Transportation of IMM. The number of vehicles crossing the bridges was obtained from IMM’s traffic measurement sensors and the General Directorate of Highways; the number of vehicles crossing through the Eurasia Tunnel was provided by the DLH Marmaray Regional Directorate. The analyses used traffic measurement data from bluetooth sensors located on the TEM Expressway, the O-6 Northern Marmara Expressway, the D100 Highway and on the coastal road on the European side (Eurasia Tunnel route).

In this work, average speeds and travel times of analysis routes were examined. The impact of the Eurasia Tunnel and the removal of additional traffic lanes on the analysis routes was studied. Changes in average speeds and travel times between Atatürk Airport and D100 Kozyatağı were calculated and compared with the D100 Highway crossing the July 15th Martyrs Bridge; the TEM Expressway crossing the FSM Bridge, and the European coastal road crossing through the Eurasia Tunnel in both directions. Changes in traffic during the morning and evening peak hours and during the daytime were analyzed. Routes used in the analyses and comparisons on D100 Highway (Figure 3), on TEM Expressway (Figure 4) and on coastal road routes (Figure 5) are shown in the figures below.
4 TRAFFIC STATUS OF THE EURASIA TUNNEL

The Eurasia Tunnel, which was opened on December 20, 2016, operated as single lane between 07:00am - 09:00pm until January 31, 2017. Full operation of all lanes without any time constraint commenced on February 1, 2017. Use of the Eurasia Tunnel as an alternative crossing between the continents is growing daily as can be seen in Figure 6 and Figure 7.
As with the July 15th Martyrs Bridge and the Fatih Sultan Mehmet Bridge, the Eurasia Tunnel is most widely used by drivers in the Asia-Europe direction during morning peak hours; and in the Europe-Asia direction during evening peak hours, as can be seen in Figure 8 and Figure 9.

5 EFFECTS OF THE EURASIA TUNNEL PROJECT

This study takes traffic measurement data of November-December 2016 as its baseline for analyzing the period before the Eurasia Tunnel opened. The post-tunnel analysis uses traffic measurement data of February-May 2017. In the analyses, traffic measurement data between 06:30am-09:00am is used for morning peak hours. For daytime, traffic measurement data between 09:00am-04:30pm, and for the evening peak hours, traffic measurement data between 04:30pm-08:00pm has been used.

5.1. Hourly Average Vehicle Counts

Figure 10 shows the hourly average number of vehicles crossing bridges and coming through the Eurasia Tunnel from Europe to Asia in February and March 2017. The number of vehicles traversing both bridges and the Eurasia Tunnel are at their highest between 05:00pm-09:00pm during evening peak hours.
5.2. Europe → Anatolia Direction Hourly Saturation Flow Rates

Figure 11 shows the hourly saturation flow rate in the Europe-Asia direction, using the HCM 2000 Vehicles/Hour/Lane value published by the Transportation Research Board (traffic data between February and March 2017). As shown in Figure 11, the observed traffic flow rate is at approximately full capacity on the July 15th Martyrs Bridge and the Fatih Sultan Mehmet Bridge at all times except at night, whereas the Eurasia Tunnel peaks at 44% of capacity.

![Europe → Anatolia Direction Hourly Saturation Flow Rate](image)

Figure 11. Europe → Anatolia Direction Hourly Saturation Flow Rate

5.3 Effects of the Eurasia Tunnel on Vehicle Counts Crossing Bridges

Figures 12 and 13 show the number of vehicles crossing July 15th Martyrs Bridge and FSM Bridge, obtained from the General Directorate of Highways. Figure 14 shows the number of vehicles crossing the Yavuz Sultan Selim (YSS) Bridge, collected from IMM’s traffic measurement sensors network. Based on these figures, the monthly average number of vehicles on the July 15th Martyrs Bridge after the opening of Eurasia Tunnel fell about 6% in the first quarter of 2017 compared with the first quarter of 2016. The total number of vehicles crossing the FSM Bridge fell by about 23% in the first quarter of 2017 compared to the first quarter of 2016 due to the use of the YSS Bridge by heavy tonnage vehicles.

![July 15th Martyrs' Bridge Yearly Vehicle Counts](image)

Figure 12. July 15th Martyrs Bridge Yearly Vehicle Counts
When the average number of vehicles making intercontinental trips in 2016 and in January-May 2017 are compared; a decrease of about 2% is observed as seen in Figure 14.

Figure 13. Fatih Sultan Mehmet Bridge Yearly Vehicle Counts

When the average number of vehicles making intercontinental trips in 2016 and in January-May 2017 are compared; a decrease of about 2% is observed as seen in Figure 14.

Figure 14. Intercontinental Monthly Average Vehicle Counts (Averages of January-May)
After the opening of the Eurasia Tunnel, no significant change was observed in the number of vehicles crossing the July 15th Martyrs Bridge, as seen in Figure 15, while the number of vehicles crossing the FSM Bridge fell slightly due to the impact of the YSS Bridge, which is seen in Figure 16.

5.4 The impact of the Eurasia Tunnel on Travel Times

Based on traffic measurement data of February-May 2017, the average travel time between Kozyatağı and Atatürk Airport, normally around 75 minutes in the morning peak hours, decreased to 40 minutes when crossing through the Eurasia Tunnel. On the same route in the opposite direction, the average travel time of approximately 85 minutes in the evening peak hours fell to 54 minutes. Thus it can be seen that drivers save up to 36 minutes during peak hours on the Kozyatağı-Atatürk Airport route.
5.5 Effects of Eurasia Tunnel to Traffic of D100 Highway

As seen in Figure 19 and Figure 20, an increment in average speeds up to 43% in the Anatolia → Europe direction and up to 63% in the Europe → Anatolia direction was observed on the D100 Highway following the opening of the Eurasia Tunnel and the removal of additional lanes during peak hours on the bridge crossings. Improvement in traffic conditions on intercontinental transits also positively affected travel times on the D100 Highway. Average travel times fell by up to 30 minutes in the Anatolia → Europe direction during the evening peak hours and by up to 26 minutes in the Europe → Anatolia direction during the morning peak hours.

### Table 1. Comparison of (October-December 2016) with (February-May 2017)

<table>
<thead>
<tr>
<th></th>
<th>Morning</th>
<th>Daytime</th>
<th>Evening</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Speed (km/h)</td>
<td>Before</td>
<td>25</td>
<td>29</td>
</tr>
<tr>
<td></td>
<td>After</td>
<td>27</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td>Average Speed (%)</td>
<td>%8</td>
<td>%14</td>
</tr>
<tr>
<td>Travel Time (min)</td>
<td>Before</td>
<td>82</td>
<td>71</td>
</tr>
<tr>
<td></td>
<td>After</td>
<td>76</td>
<td>62</td>
</tr>
<tr>
<td></td>
<td>Travel Time (%)</td>
<td>%-7</td>
<td>%-13</td>
</tr>
</tbody>
</table>

5.6 The impact of the Eurasia Tunnel on Traffic on the TEM Expressway

As seen in Figure 21 and Figure 22, an increment in average speeds of up to 29% in the Anatolia → Europe direction, and of up to 38% in the Europe → Anatolia direction was observed on the TEM Expressway following the opening of the Eurasia Tunnel and the removal of additional lanes during peak hours on bridge crossings. Improvement in traffic conditions during intercontinental transits also positively affected travel times on the TEM Expressway. Average travel times fell by up to 18 minutes in the Anatolia → Europe direction during the evening peak hours and in the Europe → Anatolia direction during the morning peak hours.

### Table 2. Comparison of (October-December 2016) with (February-May 2017)

<table>
<thead>
<tr>
<th></th>
<th>Morning</th>
<th>Daytime</th>
<th>Evening</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Speed (km/h)</td>
<td>Before</td>
<td>30</td>
<td>41</td>
</tr>
<tr>
<td></td>
<td>After</td>
<td>49</td>
<td>48</td>
</tr>
<tr>
<td></td>
<td>Average Speed (%)</td>
<td>%63</td>
<td>%17</td>
</tr>
<tr>
<td>Travel Time (min)</td>
<td>Before</td>
<td>68</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>After</td>
<td>42</td>
<td>42</td>
</tr>
<tr>
<td></td>
<td>Travel Time (%)</td>
<td>%-38</td>
<td>%-16</td>
</tr>
</tbody>
</table>
Contribution of the opening of the Eurasia Tunnel and the removal of additional lanes at bridge crossings for the optimization of the traffic on the July 15 Martyrs Bridge was higher compared to the FSM Bridge.

5.7 The impact of the Eurasia Tunnel on Traffic on the European Coastal Road

As seen in Figure 23 and Table 5, following the opening of the Eurasia Tunnel, no traffic density is observed on the European Coastal Highway on the Atatürk Airport - Samatya route. Average speeds on this route rose by up to 55% and travel times fell by up to 35%.

---

**Table 3. Comparison of (October-December 2016) with (February-May 2017)**

<table>
<thead>
<tr>
<th>Average Speed (km/h)</th>
<th>Morning</th>
<th>Daytime</th>
<th>Evening</th>
</tr>
</thead>
<tbody>
<tr>
<td>Before</td>
<td>42</td>
<td>56</td>
<td>35</td>
</tr>
<tr>
<td>After</td>
<td>41</td>
<td>57</td>
<td>45</td>
</tr>
<tr>
<td><strong>%</strong></td>
<td><strong>-2</strong></td>
<td><strong>2</strong></td>
<td><strong>29</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Travel Time (min)</th>
<th>Morning</th>
<th>Daytime</th>
<th>Evening</th>
</tr>
</thead>
<tbody>
<tr>
<td>Before</td>
<td>70</td>
<td>52</td>
<td>83</td>
</tr>
<tr>
<td>After</td>
<td>71</td>
<td>51</td>
<td>65</td>
</tr>
<tr>
<td><strong>%</strong></td>
<td><strong>1</strong></td>
<td><strong>-2</strong></td>
<td><strong>-22</strong></td>
</tr>
</tbody>
</table>

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**Table 4. Comparison of (October-December 2016) with (February-May 2017)**

<table>
<thead>
<tr>
<th>Average Speed (km/h)</th>
<th>Morning</th>
<th>Daytime</th>
<th>Evening</th>
</tr>
</thead>
<tbody>
<tr>
<td>Before</td>
<td>45</td>
<td>63</td>
<td>35</td>
</tr>
<tr>
<td>After</td>
<td>62</td>
<td>67</td>
<td>34</td>
</tr>
<tr>
<td><strong>%</strong></td>
<td><strong>38</strong></td>
<td><strong>6</strong></td>
<td><strong>-3</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Travel Time (min)</th>
<th>Morning</th>
<th>Daytime</th>
<th>Evening</th>
</tr>
</thead>
<tbody>
<tr>
<td>Before</td>
<td>65</td>
<td>46</td>
<td>84</td>
</tr>
<tr>
<td>After</td>
<td>47</td>
<td>43</td>
<td>85</td>
</tr>
<tr>
<td><strong>%</strong></td>
<td><strong>-28</strong></td>
<td><strong>-7</strong></td>
<td><strong>1</strong></td>
</tr>
</tbody>
</table>

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**Table 5. Atatürk Airport → Samatya Comparison of (October-December 2016) with (February-May 2017)**

<table>
<thead>
<tr>
<th>Average Speed (km/h)</th>
<th>Morning</th>
<th>Daytime</th>
<th>Evening</th>
</tr>
</thead>
<tbody>
<tr>
<td>Before</td>
<td>40</td>
<td>42</td>
<td>45</td>
</tr>
<tr>
<td>After</td>
<td>62</td>
<td>61</td>
<td>56</td>
</tr>
<tr>
<td><strong>%</strong></td>
<td><strong>-65</strong></td>
<td><strong>-48</strong></td>
<td><strong>-24</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Travel Time (min)</th>
<th>Morning</th>
<th>Daytime</th>
<th>Evening</th>
</tr>
</thead>
<tbody>
<tr>
<td>Before</td>
<td>23</td>
<td>22</td>
<td>20</td>
</tr>
<tr>
<td>After</td>
<td>15</td>
<td>16</td>
<td>18</td>
</tr>
<tr>
<td><strong>%</strong></td>
<td><strong>-35</strong></td>
<td><strong>-27</strong></td>
<td><strong>-10</strong></td>
</tr>
</tbody>
</table>
6 CONCLUSIONS

This paper has examined the effects of the opening of the Eurasia Tunnel and the removal of additional lanes during peak hours on traffic crossing the intercontinental bridges and on urban traffic in Istanbul. The results show that travel times have fallen by 7%–38% on the July 15th Martyrs Bridge and by 1%–28% on the FSM Bridge. Average speeds rose by up to 63% on the July 15th Martyrs Bridge and by up to 38% on the FSM Bridge during peak hours.

No significant change was observed in number of vehicles crossing the July 15th Martyrs Bridge and the FSM Bridge as a result of the opening of the Eurasia Tunnel, while traffic flow in intercontinental transits was effected positively overall. Moreover, it is observed that densities in traffic disappeared completely on the European coastal road so that smooth traffic flow was achieved.

According to the analysis results, the hourly saturation flow rate is approximately at full capacity on the July 15th Martyrs Bridge and the FSM Bridge at all times except at night; however, it peaks at 44% in the Eurasia Tunnel. Thus, some adjustments to encourage drivers to use the Eurasia Tunnel could result in improved distribution of vehicles as between the bridges and the tunnel (Figure 11. Europe→Anatolia Direction Hourly Saturation Flow Rate).

The average travel time on the D100 Kozyatağı → Atatürk Airport route – 100 minutes before the opening of the Eurasia Tunnel – fell by up to 30 minutes during peak hours after it opened.

The Eurasia Tunnel has cut the time of intercontinental transits by increasing the alternatives for drivers. It has lowered the cost of travelling time by increasing transport capacity and resulted in a more comfortable travel experience. The reduction in bridge crossing times has also started to bring down the associated costs of tiredness, stress and loss of business productivity that are caused by traffic delays. 4

Thanks to the Eurasia Tunnel Project, travel times are reduced with corresponding falls in fuel consumption and vehicle maintenance costs, thus contributing to the national economy. The project has enabled Istanbul to achieve a green transportation infrastructure that does not damage the city's natural beauty and skyline, or the aquatic life of Bosporus, while also protecting ecological balances. The link between the two airports provided by the tunnel will make a significant contribution to Istanbul’s position in international air transportation, since it is the most practical route between Atatürk Airport in Europe and Sabiha Gökçen Airport on the Anatolian Peninsula. 7

7 REFERENCES

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**PAPER TITLE**
Chaos and Disruptions as the Challenge to Urban Transportation in Tanzania

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No. 37

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Chaos, Disruptions, Urban, Transportation Tanzania

**ABSTRACT:**
Transportation is a key to the economy and production; it makes mobility more accessible and enhances the social and economic interactions. On the other hand, the increase of urban population, pollution and other negative impacts directly compromise the existing transportation systems and endanger the future transportation systems in developing countries. This paper examines chaos as the challenge facing urban transportation in Tanzania cities and provides some suggestions to reduce the existing problem. This has been done by looking at the design and plan of the Tanzania cities, coordination of transportation systems and car dependency. Environmental and social impacts which include congestion's, air pollution, traffic accidents and energy consumption have been described. Suggestions for addressing the challenges facing urban transportation in developing countries like Tanzania have been examined by adopting the holistic approach. Such approach has shown to be effective in solving the challenges facing urban transportation in the cities of developing countries such as improving public transport, provision of off-street parking, enforcement of traffic laws and regulations and restrict car use. Moreover, approaches to alleviate challenges facing urban transportation should be designed for specific cities and urban transport planners must understand that models and solutions used for cities in the developed countries may not be applicable to cities of developing countries.
Chaos and Disruptions as the Challenge to Urban Transportation in Tanzania

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Email: brunokinyaga@gmail.com

1.0 Introduction

Problems with urban transportation is a major challenge in both developed and developing countries since it interlinks with most (if not all) sectors of the urban setting. Per a World Bank study, the challenges of urban transport have been associated with globalization, urbanization, fiscal decentralization and economic transition. The growth of the population and density of the buildings in the cities only add further to the difficulties of traffic and plague to endless congestion, grave air pollution, alarming accident rates and lengthy travel time to work (Drakakis-Smith, 2003).

The population of many cities in Tanzania has grown to the extent that cannot be controlled easily in recent years, and this increase is expected to continue in the foreseeable future. Fast-growing cities in Tanzania such as Dar es Salaam, Mbeya, Mwanza, Arusha and others have nurtured business and industries, which have resulted in creating jobs and higher income to migrants from rural areas. Thus, this increase has changed the morphology of the cities and enlarged the challenges in the urban transport systems, have resulted in congestion and delays in both passenger and products from the different places to reach the market; high level of pollution, fatalities and injuries have been increased (Mrema, 2011). Statistical report shows that about 70 percent of vehicles registered in Tanzania remains in the growing cities (TRA, 2010.

Dar es Salaam Rapid Transit (DART) indicates that about 4 billion Tanzanian Shillings lost every day in the city due to challenges associated with urban transportation. Road traffic accidents (RTAs) in Tanzania cities are estimated on average to cause 3400 deaths per annum and about 20000 serious injuries (Nyambacha, 2011).

The purpose of this paper is to examine the challenges facing urban transportation in Tanzania, using a holistic approach which considers the relationship of land management and transportation as a dynamic system involving other factors other than congestions.

Figure 1. Tanzania Map
2.0 Literature Review

Urban transport is the movement of people and goods within urban areas using the technologies such as marines, roads and rails transportation systems. The challenges of urban transportation occurring in the urban cities are the result of globalization, urbanization, fiscal decentralization and economic transition. The notable challenges facing urban transport include, long commuting, traffic congestion and parking difficulties, the inadequacy of public transport, difficulties for non-motorized transport, loss of public space, accident and safety, environmental impacts and energy consumption, land consumption and freight distributions. Location of the cities comprises different levels of accommodation and concentration of economic activity, which is pronounced to be among the complex structures that are supported by transportation systems (Rodrigue, 2009).

The notable urban transportation problems arise when when the city is large consists of complex structures, and the potential of disruption is very high. If this complexity is not well managed transportation infrastructures cannot meet the requirement of the demand for urban mobility. This is the major challenge to the transportation systems and inefficiency of the systems (Banks, 2002).

Hence transportation is the hub most growing cities to enhance productivity and control the economy; hence effective and efficiency measures must be employed before the changes have resulted in severe damage because the movement of labour, consumers and freight from origin to destination depends on the effectiveness of the transportation systems.

3.0 Urban Transportation in Tanzania

Urban transport in Tanzania is predominantly a road based, motorized and non- motorized. Other modes include rail and water based, which is not yet developed. Tanzania has a national transport policy since (2003) regulated by different authorities (Ministry of communication and transport, the ministry of finance controlling motor vehicle registration, regional road administration and planning commission), although little attention is given to urban transport issues.

The policy manages the urban roads and other infrastructure, road services, traffic flow and management, and land- use planning and transport for disadvantaged groups. However, pedestrians and non-motorized are not considered during the implementation of policies. It is necessary to consider this group because often are the losers in the struggle for available space and have no power to influence the urban transport policies.

Recently, the stakeholders and authorities involving transportation planning have combined effort towards building way out of congestion by increasing road width in urban cities. Although stakeholders of transport do recognize some challenges with traffic congestion, and the impact that poor land use planning has on the traffic flow and congestion yet they must decide to proceed with implementation of widening most of the roads in cities of Tanzania.

It is estimated that most daily trips in urban cities like Dar- es-Salaam are using public transport (61%), while only (10%) take private cars and the remainders are through walking and bicycling. However, public transport which serves many people is not given any attention in Tanzania.

There is a new policy in Tanzania currently being under the assistant of the Department for International Development (DFID) which supports the technical assistance program within the ministry of transport. Under this program, more attention has been given to public transport, Bus Rapid Transit (BRT).
4.0 Main Causes of Disruptions in Tanzania

Tanzania is among the developing countries with rapid urbanization and fast-growing cities. A study indicating the changing of the morphology of many Tanzania cities gives an overview of the challenges of urban transportation in Tanzania. The problem of chaos and disruptions in cities of Tanzania is argued to be caused by behaviour of drivers/users of the roads, road/vehicle conditions, population growth, the design of the cities and limited flow capacity (Bundara, 2010).

4.1. Vehicular growth: Statistical data available in the Tanzania Revenue Authority (TRA) in the department of vehicle registration indicate that the growth rates of vehicles have reached 15 percent during the year 2016. The growth rates in the cities like Dar es-Salaam for the past 20 years were 8 percent while in the other urban, city was 2 percent. The number of vehicles in the country is estimated to be 1.5 million, 80 percent are in urban cities. Dar es-Salaam constitutes many the vehicles with 70 percent while the remaining vehicles are in the other cities.

Dar es Salaam with 5 million of Tanzania populations is estimated that there are 40 vehicles in every 1000 people with 1800 square km. The problem is not the number of vehicles in the country but the concentration of the vehicles in few selected cities, particularly in the urban cities. Therefore, in the absence of an adequate and efficient public transport system, many the private and mixed transit modes have entered and will continue to enter the market to meet the travel demand. The increase of the vehicle, city results into acute congestion and delays, serious accidents, high energy consumption and intense pollution.

Table 1: Dar es Salaam Estimated number of Trips per day per mode in 2016

<table>
<thead>
<tr>
<th>Mode</th>
<th>Number of vehicles</th>
<th>Average distance travelled</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dala Dalas</td>
<td>8000</td>
<td>10 Km</td>
<td>61%</td>
</tr>
<tr>
<td>Walking &gt;500 M</td>
<td>-</td>
<td>2km</td>
<td>24%</td>
</tr>
<tr>
<td>Passenger Cars</td>
<td>130,000</td>
<td>15Km</td>
<td>15%</td>
</tr>
<tr>
<td>Bicycles</td>
<td>200,000</td>
<td>5Km</td>
<td>2%</td>
</tr>
<tr>
<td>Motorcycles</td>
<td>40,000</td>
<td>10Km</td>
<td>2%</td>
</tr>
<tr>
<td>TOTAL</td>
<td>378,000</td>
<td></td>
<td>100%</td>
</tr>
</tbody>
</table>

Traffic jam is caused mainly by the 130,000 (and rapidly increasing) passenger cars which satisfy only 15% of transport demand.
4.2 Parking difficulties: It is pronounced to be another reason associated with chaos and disruption. Roadside and unlawful parking are common features in Tanzania, especially in the CBD which forces some people to park in roadside, thus why the road becomes even narrower (Kiunsi, 2011). The ineffective regulation of parking has accelerated to worsen the situation. In the CBD, vehicles spend a lot of time in a parking which has increased demand of land consumption. Even when the parking facilities are provided, but the demand for parking is very high since there is an increase in motorization (Kiunsi et al, 2006).

![Image](image1.jpg)

Figure 2: Kariakoo Business Street, Dar es Salaam,

4.3 High frequency of Accidents in Tanzania urban centers: Urban environment is the most prone area of motor accidents; it is estimated that on average in Tanzania, about 3400 deaths occur for each year and 20000 serious injuries in the major cities of Tanzania (Tanzania Traffic Police Force). The situation has been contributed by undue concentration of vehicles in urban areas, traffic mix and resultant flow conflicts. Most of these accidents happen due to the general impatience and ill-tempered nature of road users and the conflict between motorcycle, pedestrians and other users of road transport in the cities.

4.4 Existing infrastructures: The road space in Tanzania is insufficient. Most of major roads and junction in Tanzania are crowded with parking vehicles, roadside hawkers and pavement dwellers. Thus, the roads for moving vehicles become much narrower resulting to chaos and disruption in cities. Currently, in Tanzania, the inner-city rail service operates only in Dar es-Salaam. Other urban cities use a bus and other non-motorized as a means of transport. Most of the roads are not in good conditions and the buses carrying passengers in the urban cities are not specifically designed for urban conditions. The buses (Daladala) operating in urban cities of Tanzania are overcrowded, unreliable, not safe and involve long waiting. Overcrowding in the public transport is pronounced to be one of the reasons for passengers to shift to personalized transport.
Other urban transportation challenge in Dar es Salaam is on its lack of resilience against the effects of climate change such as flooding.

**5.0 Possible Solutions for the Urban Transportation Problems in Tanzania**

The following specific measures are to be taken for improving urban transportation in Tanzania.

- **Road Infrastructure Improvements**, In Sustainability more recent years an understanding has emerged that increasing capacity can lead to greater demand as a result of “induced travel” (also referred to as “latent demand” or “generated traffic”). Induced travel is due to diversion of travel from: (1) other lower volume hours of the day to more peak hour use of improved facilities; (2) parallel commuting routes; and (3) public transportation.

- **Improve Road-Based Public Transport**, Effective road-based public transport is central to economic growth of developing cities. For the majority of residents, road-based public transport (bus and paratransit) is the only means to access employment, education, and public services. In medium and large developing cities, such destinations are beyond viable walking and cycling distances while vast numbers of individuals have limited access to automobiles.

- **Technological Solutions**, The main urban transport-related technological solutions that cities worldwide are currently pursuing (with North American and West European cities leading the way) include alternative-fuel vehicles and intelligent transportation systems (ITS). New technologies may help to tackle certain transport-related problems, such as air and noise pollution, oil dependency, traffic congestion, and accidents.

- **Awareness-Raising Campaigns**, Developed and developing countries have used information, education, persuasion, and awareness-raising campaigns in favor of sustainable urban transport with various, but generally limited, degrees of success. Typically, the more effective a measure is, the more resistance it evokes. Social mechanisms and processes, such as status seeking (i.e., the automobile as a status symbol), freedom seeking, or lack of trust in others’ cooperativeness, are often at play, especially in the developing world, and perpetuate urban transport problems.
- **Provision of traffic light at major junction of the cities.** Larger volumes of the traffic are observed during the peak hours almost in every outstanding road of the cities in Tanzania. Other roads should be provided with “STOP” sign at appropriate arm junctions, and others should be managed by traffic wardens accordingly. In addition, all roads in Tanzania cities should be provided with road signs whenever there is no sign.

- **Control of Land Uses,** Generally public transport and non-motorized modes require high densities and mixed uses in order be practically and financially feasible. Compact urban development is also often associated with shorter distances and lower use of motorized transport. Therefore, land-use controls have important implications for travel behavior. In smaller cities in particular, the manipulation of urban form (shape, size, density, compactness, intensification, decentralization, land-use type and mix, building layout and type, and green and open spaces) can help to overcome city problems.

- **Regular maintenance of roads in cities.** The road maintenance agencies should be well funded to carry out their duties. The government must pay attention during the rehabilitation of the major roads. Furthermore, whenever there is the largest concentration of pedestrian’s complete separation of vehicles should be encouraged to reduce pedestrians-vehicular conflicts in the cities. This can be achieved by creating barriers such as underpasses and overhead foot bridge.

- **Intensive studies of Transportation problems:** There has not been any comprehensive transportation study for many urban centres in Tanzania. Thus, the volumes of traffic along many urban routes of cities are not known. A time series data on the various components of urban traffic is important to city planners interested in future transportation planning.
7.0 Conclusion and Recommendations

The challenges of the chaos and disruptions in urban transport cannot be solved without clear coordination of stakeholders together with suitable policies. Urban area, whether big cities, cities or town has grown and will continue to expand, but the demand has always exceeded the level of service provided. The deteriorating of public transport forces people to shift to personalized transports, which are not safe, fuel-inefficient, increase traffic congestion and increase pollutions.

In Tanzania, it shows clearly that multitude of stakeholders both (formal and informal), layers of geographical inversions and competencies and the intersection between policy formulation, regulation, service provision and service user, reveals a high level of complexity, contradiction and overlaps. Although there are many stakeholders with some level of responsibility towards ensuring a functioning of the urban transport system, there is no clear institution that coordinates or is accountable for engage in the whole process of the urban transport problem because of the overall inefficiency in rendering the service.

This chaotic urban transport system must be reorganized into an efficient and sustainable system that prioritizes public transport and non-motorized transport.

This paper has examined the nature, type and causes of chaos and disruptions of urban transportation in Tanzania cities and has made some possible suggestions to reduce the problems. However, urban transportation remains to be challenging phenomena recurs in many urban centres, combined efforts should be to adopt “Best Practices” which has shown to be effective in tackling the transportation problems in developing countries like Tanzania. It is suggested that approaches that are efficient and flexible is one needed by developing countries to alleviate the transportation problems occurring in various urban cities, and the finest way is for every city to develop its own version and models to examine the challenges facing transportation systems.
References


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Shockwave Dampening

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Abstract

The purpose of applying shockwave theory is to manage congestion by balancing out inflow and outflow of traffic at bottleneck. Compare traffic flow to a granular flow through a narrow bottleneck like hour-glass. If traffic demand at a certain location is larger than capacity the congestion will occur.

Traffic shockwaves are the transition zones between two traffic states that move through a traffic environment like a propagating wave. The sudden transition between two traffic states sends a propagating wave backward initiating the jam.

The theory behind Shockwave Dampening is to recognize and manage the issue before the propagating wave develops by reducing the volumes of approaching traffic so that inflow of vehicle will be equal to or less that outflow of vehicles passing through the bottleneck. The upstream traffic stream may be managed by using two controls:

- **Ramp Metering:** using traffic signals to control on-ramp volumes entering the traffic corridor
- **Dynamic Speed Control:** electronic messaging signs upstream will modify the speed limit of the traffic corridor to slow down upcoming traffic thus reducing traffic flow rate.

Shockwave Equations can be used to determine the speed and flow rate of the traffic passing through the bottleneck (Outflow). The speed and flow rate of upcoming traffic on the main highway and ramps (inflow) can be obtained through traffic cameras. After determining the speed and flow rate of the congested section of the roadway, the flow rate of the upstream traffic will be reduced in order to match this flow rate with that of the congested section. The upstream flow rate of the mainline traffic is reduced by reducing the mainline speed gradually through dynamic traffic control using variable message signs. The flow rate of the on-ramps will be reduced by installing ramp meters with two-phase signals; stop/go to control the influx of the traffic flow onto the mainline.
Introduction

A shockwave in traffic stream describes the boundary between the two traffic states that are characterized by different speeds, densities and flowrates. Shockwave theory is able to describe the spatio-temporal properties of queues more accurately comparing with queuing theory. Shockwaves are a boundary that shows discontinuity in a flow-density domain.

The purpose of this paper is to test the potential of the shockwave dampening tools which may be used to prevent shockwaves from happening by using a VISSIM simulation model. These techniques were used to prevent discontinuity in traffic flow and density thus eliminating / minimizing the chance of shockwaves propagation in the traffic stream.

Fundamentals of the Shockwave Theory

The physical realization of a shockwave is a point in time and space at which vehicles change their speed abruptly. Shockwaves are the transition zones between two traffic states that move through a traffic environment like a propagating wave. The sudden transition between two traffic states sends a propagating wave backward initiating a traffic jam.

The traffic flow can be compared with a granular flow through a narrow bottleneck like hour-glass. If traffic demand at a certain location is larger than the capacity of roadway at that location the congestion will occur. A shock wave propagates along a line of vehicles in response to changing conditions at the front of the line.

For example, as shown in the below illustration, the capacity of a roadway is reduced due to a construction work. The flow rate and density of the traffic stream will abruptly change at the boundary of the construction area. This will generate the backward forming shockwaves which will continue to grow until the upcoming flow rate is equal to or less then the flow rate of the bottleneck section.

Shockwaves can be generated by a number of means like reduction in the roadway capacity, collisions or sudden increases in speed caused by entering free flow conditions.

Shockwave Equation

The equation that is used to estimate the propagation velocity of shock waves is given below.

\[ w = \frac{(qa - qb)}{(ka - kb)} \quad \ldots \ldots \ldots (1) \]

Where:

\( w \) = propagation velocity of shock wave (km/hour)
qb = flow prior to change in conditions (vehicles/hour)
qa = flow after change in conditions (vehicles/hour)
kb = traffic density prior to change in conditions (vehicles/km)
ka = traffic density after change in conditions (vehicles/km)

The shockwave speed is measured at a particular interface or a boundary where a free-flow condition is mixing with the jammed traffic (where capacity is reduced), and ‘a’ and ‘b’ denotes jam (congestion) and free-flow conditions respectively. Using the fundamental relationship between the variables, the ‘k’ can be represented as:

\[ k = \frac{q}{u} \]  

(2)

where ‘u’ is the speed.

Substituting ‘k’ into Equation (1) and solving for ‘qa’:

\[ w = qa - qb \left( \frac{qa}{ua} - \frac{qb}{ub} \right) \]

(3)

To estimate ‘w’, ‘ua’ and ‘ub’, a breakpoint speed has to be selected. Breakpoint speed is a point where traffic transitions from a free-flow to a congestion state. After selecting a breakpoint, any speed observations greater than breakpoint speed is considered to be free-flowing and any observations smaller than breakpoint speed is considered under congestion. ‘ua’ and ‘ub’ are the average speed for the congested and free-flow regions respectively.

**Shockwave Classifications**

From the shockwave equation, the following can be noted:

If qb < qa, the shockwave speed is negative which indicates that it is backward forming shockwave.

If qb > qa, the shockwave speed is positive which indicates that it is forward recovery shockwave.

If qb = qa, the shockwave speed is zero which indicates that it is backward stationary shockwave

**Formation and Dissipation of Shockwaves**

The formation and dissolving of congestion are phenomena that are important for the traveller’s information and congestion management perspectives.

If the capacity of a roadway is reduced due to any reason, the traffic flow rate will reduce and the density will increase within the congested section. Substituting these values into the shockwave equation yields a negative propagation velocity. This means that the shockwave is traveling against the traffic and it is called a backward forming shockwave.

However, at certain time segment, the demand of the upstream traffic will start reducing due to any reason like diversion of traffic to alternate routes or starting of a non-peak hour traffic demand. At a certain instance during
this time segment, the upstream traffic flow rate will be equal to the flow rate through the bottleneck section resulting in a backward stationary shockwave.

As the upstream traffic flow demand is further reduced, the shockwave will transform to a forward recovery shockwave and will dissipate at certain instance of the time.

**Flow-Density and Time-Distance Diagrams**

The flow-density curve is shown in Figure 1.

![Shockwave: Flow-Density Curve](image)

**Figure 1:** Shockwave: Flow-Density Curve

A shockwave is basically a movement of the point that demarcates the two traffic stream conditions. As shown in Figure 1, the speed of the vehicles at state A is given by the line joining the origin and point A and the speed of the vehicles at state B is the line joining the origin and point B of the flow-density curve in the graph. The red line in the above figure represents the shockwave and the slope of the line A-B gives the speed of the shockwave.

The time-space diagram of the traffic stream is also plotted in Figure 2.

![Shockwave: Time-Distance Diagram](image)

**Figure 2:** Shockwave: Time-Distance Diagram

As shown in Figure 2, all the lines at state A are having the same slope which implies that they are moving with a constant speed. The sudden change in the characteristics of the stream leads to the formation of a shock wave. There will be a cascading effect of the vehicles in the upstream direction. This is clearly marked by red line in Figure 1. Thus the shockwaves produced at state B are propagated in the backward direction.
Shockwave Dampening Techniques

The purpose of applying the shockwave dampening techniques is to prevent shockwaves from propagating in the traffic stream and manage congestion by balancing out inflow and outflow of traffic through the bottleneck condition.

The theory behind shockwave dampening is to recognize and manage the issue before the propagating wave develops by reducing the flow rate of the approaching traffic so that upstream traffic flow rate will be equal to or less than the flow rate passing through the bottleneck condition.

The upstream traffic stream may be managed by using two controls as shown in the below illustration:

- **Ramp Metering**: using traffic signals to control on-ramp volumes entering the traffic corridor
- **Dynamic Speed Control**: electronic messaging signs upstream will modify the speed limit of the traffic corridor to slow down upcoming traffic thus reducing traffic flow rate.

In case of a planned construction activity or any other event which requires closing of a roadway lane, the Shockwave equations can be used to estimate the speed and flow rate of the bottleneck condition (congestion) in advance. As mentioned earlier that the ‘qa’, ‘ua’ and ‘ub’ are dependent upon breakpoint speed. Since these variables are used in Equation (1), they also affect the results for ‘qb’. Hence, it is important to select a proper breakpoint speed in order to have a good estimation of ‘qa’ and ‘qb’.

In case of an accident, a video-based vehicle detection system using video camera can be used to determine the speed and flow rate through the congested section as well as the speed and flow rate of upstream traffic (free-flow condition) on the mainline. The video cameras may be mounted on emergency vehicles which may be dispatched immediately to the accident site to obtain the above information.

After determining the speed and flow rate of the congested section of the roadway, the flow rate of the upstream traffic will be reduced in order to match this flow rate with that of the congested section. The upstream flow rate of the mainline traffic is reduced by reducing the mainline speed gradually through dynamic traffic control using variable message signs. The flow rate of the on-ramps will be reduced by installing ramp meters with two-phase signals; stop/go to control the influx of the traffic flow onto the mainline.

The reduction in speed of the mainline traffic and cycle length of the on-ramp meters should be applied in such a way that the average delay per vehicle should be almost equal for both the mainline and on-ramp traffic.
VISSIM Simulation Model

To test the theory of the shockwave dampening, a three (3) kilometre long segment of a main traffic corridor (Sheikh Zayed Road) in Dubai has been simulated in a VISSIM model. Only the northbound direction is simulated from Hessa Street to Umm Suqeim Street. The segment is a six-lane free flow facility with a two-lane on-ramp from Hessa Street. The two merging lanes are tapered back to the basic six-lane section before an off-ramp to Umm Squeim Street / Mall of Emirates. The location of the study section is shown below:

The upstream through traffic is 7,940 vehicles per hour and on-ramp traffic is 2,880 vehicles per hour resulting in 10,820 vehicles per hour in congested section. At downstream, the through traffic is 9,150 vehicles per hour and off-ramp traffic is 1,670 vehicles per hour.

Seven traffic counters were installed in the VISSIM model to determine the speeds and flow rates at upstream, downstream and within the congested area. The location of counters, traffic volumes and geometry of the simulated section is shown in Figure 3.

The study section experiences a high level of congestion due to heavy volumes, lanes reduction and the short weaving section. Shockwave dampening techniques were applied to remove/reduce congestion and improve flow rate and speed at the study section.

The screenshot of the VISSIM model for base condition (without improvement) is shown in the below illustration. It can be observed that a large number of vehicles are shown with red or pick colours which represent low traffic speeds (0 to 20kph). This indicates that the speed on mainline is drastically reduced in the bottleneck section.
In the improved scenario, the speed of upstream traffic is gradually reduced from 100 kph to 80 kph and then 60 kph in 3km section upstream of the study section. For ramp meter, one-vehicle-per-green operation was considered with 3 seconds for green and 3 seconds for red (6 seconds cycle length).

The screenshot of the VISSIM model for improved condition is shown in the below illustration. It can be observed that a large number of vehicles changed their colours from red/pink to light green/dark green which indicates that after applying shockwave dampening techniques the speed of vehicles in the bottleneck section were drastically improved from 0kph-20kph range to 50kph-80kph range.

Results

The results of the VISSIM simulation model using shockwave dampening techniques were very promising. The measured speeds at the counter locations are significantly increased within the congested section and the delays are reduced considerably. The results of the VISSIM model simulation before and after applying the shockwave dampening techniques are show in Table 1 and Table 2 respectively.

<table>
<thead>
<tr>
<th>Location No</th>
<th>Speed (Kph)</th>
<th>Volume</th>
<th>Relative Delay%</th>
<th>Density</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>78</td>
<td>7,940</td>
<td>10%</td>
<td>102</td>
</tr>
<tr>
<td>2</td>
<td>71</td>
<td>7,862</td>
<td>17%</td>
<td>110</td>
</tr>
<tr>
<td>3</td>
<td>25</td>
<td>10,691</td>
<td>72%</td>
<td>429</td>
</tr>
<tr>
<td>4</td>
<td>22</td>
<td>10,533</td>
<td>75%</td>
<td>474</td>
</tr>
<tr>
<td>5</td>
<td>21</td>
<td>10,462</td>
<td>77%</td>
<td>499</td>
</tr>
<tr>
<td>6</td>
<td>64</td>
<td>9,150</td>
<td>28%</td>
<td>164</td>
</tr>
<tr>
<td>7</td>
<td>39</td>
<td>2,880</td>
<td>34%</td>
<td>75</td>
</tr>
</tbody>
</table>
As the counter 4 was installed at the middle of the congested section (refer Figure 3), it shows very significant improvements. The speed is increased from 22 kph to 58 kph and relative delay is reduced from 75% to 2%. The improvement in speed and delay is observed at all the counter locations except at the on-ramp location (counter 7) where the speed was reduced from 39 kph to 3 kph and relative delay was increased from 34% to 95%. These observations at on-ramp location were expected as vehicles are controlled through ramp meter and are operated as one-vehicle-per-green operation.

The results of the VISSIM model show an interesting pattern. At base condition, the speed of upstream traffic at counter 2 is 71 kph which is drastically reduced to 25 kph at the boundary of the bottleneck condition (breakpoint speed). The speed through the congested area is 22 kph.

After applying the shockwave dampening techniques, the speed of upstream traffic at counter 2 is reduced from 71 kph to 58 kph (because posted speed is reduced to 60 kph) but the relative delay is also reduced from 17% to 2% due to less congestion at bottleneck section. The speed through the congested area is drastically improved from 22 kph to 58 kph and delay is reduced from 75% to 2%.

Another importation point to be noted is that in the improved scenario the speed and delay are almost constant within the study section as the densities and flow rates are equal at both the upstream and bottleneck sections. This demonstrates that the shockwaves did not propagate after applying the dampening techniques.

The graphical representations of the speeds and relative delays at all the counter locations for both the base and improved conditions based on the VISSIM simulation are shown in the below illustrations.
A system wide comparison was also made to determine the overall improvement of the system. The results of the system wide comparison are shown in Table 3.

Table 3: System wide Comparison – Before and After of Dampening Techniques

<table>
<thead>
<tr>
<th>Status</th>
<th>Ave. Delay (sec)</th>
<th>Ave. Stops</th>
<th>Ave. Speed (kph)</th>
<th>Total Travel Time (hour)</th>
<th>Total Delay (hour)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Before</td>
<td>94</td>
<td>1.7</td>
<td>52</td>
<td>2,412</td>
<td>509</td>
</tr>
<tr>
<td>After</td>
<td>23</td>
<td>1.1</td>
<td>51</td>
<td>2,115</td>
<td>22</td>
</tr>
</tbody>
</table>

The system wide comparison shows that the average delay is reduced from 94 seconds to 23 seconds. The total system wide travel time is reduced from 2,412 hours to 2,115 hours and the total delay of the system is reduced from 509 hours to 22 hours. The average speed of the system is almost same in both scenarios as the posted speed is reduced from 100 kph (base condition) to 60 kph (improved condition).

Conclusions

Shockwave dampening techniques are useful techniques to prevent shockwave from propagating in traffic stream. These are based on managing congestion by balancing inflow and outflow of traffic through the bottleneck condition. These techniques manage the issue before the propagating wave develops by reducing the flow rate of the approaching traffic so that upstream traffic flow rate will be equal to or less than the flow rate through the congested area.

A three-kilometre long segment of a main traffic corridor (Sheikh Zayed Road) in Dubai is simulated in VISSIM model. This paper examines the results of VISSIM model before and after applying the shockwave dampening techniques. The results show that the speeds are significantly increased and the delays are considerably reduced within the congested section after applying the dampening techniques.

Though the system wide speed and delay are significantly improved on mainline, the on-ramp traffic from Hessa Street are held back for longer period due to installation of the ramp meter. Thus the ease of the on-ramp traffic was sacrificed for the ease of the mainline traffic.

While the results look promising, there are some limitations to this paper. First, the techniques are only tested in VISSIM model and testing the data in the field may be more complicated and may generates different results.
Secondly, the driver’s behaviour and car following models was based on the VISSIM default values which may be different in location conditions.

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Impact of Fast Development Growth on Urban Mobility

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Abstract

Over the last two decades, cities in the Middle East have witnessed significant growth in economic development resulting in population growth and higher levels of employment which in turn resulted in increased daily trips on limited transport infrastructure. In these fast developing cities where growth in land use development outstrips transport infrastructure provision there is little time for planning studies or efficient integration between land use and transport. Typically, under these circumstances the authorities tend to focus on road building rather than expanding and promoting mass transit as it is quicker to implement roads to cater for the expected travel demand. However, this approach would inevitably leads to traffic delays and lower level of urban mobility.

This paper aims to discuss the impact of fast development growth on urban mobility by comparing travel times on selected routes in Dubai. The paper also proposes measures that could contribute to maintaining acceptable level of urban mobility.

1 Introduction

Mobility is defined as the ability to move or to be moved freely. In the context of urban transport, mobility may be defined as the ability to move freely from one location to another for the purpose of satisfying a specific activity. Urban movements are therefore linked to specific activities that are related to various land uses distributed across the urban area. In an urban area, it is therefore essential to maintain an acceptable level of mobility for movement of individuals and goods through the transport network. Poor urban mobility would inevitably results in delays and longer travel times impacting economic growth.

In developed cities, the responsible authorities give particular emphasis to promoting mass transportation and efficient integration between land use and transport with a view of maintaining acceptable level of urban mobility. Numerous planning studies are undertaken to develop appropriate transport plans to move individuals and goods with emphasis on collective and sustainable transport modes.

In fast developing cities where growth in land use development outstrips transport infrastructure provision there is little time for planning studies or efficient integration between land use and transport. Typically, under these circumstances the authorities tend to focus on road building rather than expanding and promoting mass transit as it is quicker to implement roads to cater for the expected travel demand. However, this approach would inevitably leads to traffic delays and lower level of urban mobility.

2 Recent Trend

Over the last two decades, cities in the Middle East have witnessed significant growth in economic development resulting in population growth and higher levels of employment which in turn resulted in increased daily trips on limited transport infrastructure. To cope with the fast pace of development, the concerned authorities accelerated transport infrastructure provision but the pace of expansion has not matched the pace of development growth mainly due to the time required to plan and implement transport infrastructure. Furthermore, the development planning is generally led by major developers rather than through an integrated city-wide master plan. As a result, unacceptable level of traffic congestion is present, particularly during peak periods, in these fast developing cities resulting in decreased level of urban mobility.
The efficient planning of transport infrastructure requires a robust and reliable transport model to assist planners in making informed decisions. The authorities began the development of comprehensive multi-modal transportation models using the latest techniques and software. Although these models are useful in predicting future travel demand they are strategic in nature and not appropriate for addressing existing traffic and urban mobility issues. Furthermore, in a fast developing city, such as Dubai, the pattern of land use is continuously changing and land use planning is dictated by major developers. This trend makes it difficult for planners and engineers to prepare appropriate long term transport plans. There is therefore a need to also focus on current and short term traffic related issues with a view of developing appropriate traffic operation plans for maintaining acceptable level of urban mobility.

3 Recent Growth

Dubai is probably the most dynamic and rapidly growing city in the Middle East. Over the last decades, Dubai has improved its international status as a global city attracting millions of tourists annually as well as being a hub for the region and international air traffic between Europe and Asia. Furthermore, Dubai attracts significant daily traffic from the neighboring Emirates in the UAE.

Dubai’s phenomenal growth has resulted in significant increase in population, employment and tourism which in turn contributed to densification of existing urban areas, development of new land uses and provision of additional transport infrastructure. The population of Dubai increased from around 1 million in 2000 to about 2.7 million in 2016 (an increase of 170%). In parallel, many mega developments were constructed to provide housing, offices, hospitals, schools, hotels and other land uses to cater for the needs of residents and visitors. The transport infrastructure was also significantly improved to provide additional capacity and connect the new attraction centers in the city.

The transport infrastructure improvements included provision of new roads where lane kilometers increased from 8,715 in 2006 to 13,594 in 2016 (an increase of 56%). In addition to road improvements, two Metro lines covering 74.6 kilometers and 49 stations were constructed and started operation in September 2009. Also, a new tram line of 10.6 km with 11 stations was constructed and started operation in November 2014. Furthermore, many other improvements such as new bus routes, water taxis, etc. were added to Dubai’s transportation system.

Figures 1 and 2 show Google Earth images of Dubai in 2006 and 2016, respectively, giving an indication of land use densification and transport infrastructure expansion over this period. These figures clearly show the densification and development of urban areas across Dubai as well as expansion of the transport infrastructure.
Figure 1 – Dubai in December 2006

(Source: Google Earth, December 2006)

Figure 2 – Dubai in December 2016

(Source: Google Earth, December 2016)
4 Measuring Urban Mobility

Generally, in a rapidly growing city the pace of development growth outstrips transport infrastructure provision due to a number of factors including the time that it takes to plan and implement a transport scheme and the cost associated with it. There is, therefore, a risk that rapid development growth reduces urban mobility. In Dubai, particularly, numerous large-scale developments have been implemented or are under construction that generates high level of trips, and in some locations, across relatively small area. For example, in Downtown Dubai, the world’s biggest shopping mall (Dubai Mall) and tallest building (Burj Khalifa) as well as numerous high-rise residential, office and hotel buildings are located within an area of 2 km$^2$. Similarly, in Dubai Marina, there are numerous mixed-use high rise buildings and various waterfront attractions that generate high level of trips. Although, Dubai’s Roads and Transport Authority (RTA) has significantly improved Dubai’s transport infrastructure traffic congestion is present on major roads and in most dense areas during peak periods.

In order to measure the impact of development growth on urban mobility in Dubai travel times along a number of alternative routes were compared between 2004 and 2017. In April 2004, a number of travel routes were surveyed for the purpose of recording travel times along these routes for use in the development of a transport model that was used for the planning of Dubai Metro Red and Green lines. Figure 4 shows the 7 routes that were surveyed in 2004. In July 2007, 4 of the 7 routes (3, 4, 5 & 6) were surveyed again with a view of measuring the change in the level of mobility between these years. It is worth noting that although the month during which the 2017 surveys were undertaken is different from the 2014 surveys with regards to seasonal variation including school holidays as well as some modification to the road network, the comparison is still meaningful and in order to minimize the effect of school holidays only travel times for the PM peak period were compared.

The surveyed routes cover some of the key roads in Dubai particularly along the Metro corridor. Each route was surveyed twice during the PM peak periods using the moving car observer method, where a driver drives the car down the route at as close as possible to the prevailing speed in the traffic flow, while observing the need to drive safely at all times. Table 1 shows travel times for the 4 surveyed in 2004 and 2017.
Figure 4 – Travel Time Routes along Dubai Roads

Table 1 – Average Route Travel Time during PM Peak Period (17:00 to 20:00)

<table>
<thead>
<tr>
<th>Route</th>
<th>Direction - From</th>
<th>Direction - To</th>
<th>2004 (minutes)</th>
<th>2017 (minutes)</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>Baniyas Road to Jaddaf</td>
<td>Jaddaf to Baniyas Road</td>
<td>31.6</td>
<td>43.3</td>
<td>37.0</td>
</tr>
<tr>
<td>3</td>
<td>Jaddaf to Jaddaf</td>
<td>Baniyas Road</td>
<td>34.2</td>
<td>63.4</td>
<td>85.4</td>
</tr>
<tr>
<td>4</td>
<td>Sharjah Border to Jumeirah Road</td>
<td>Jumeirah Road to Sharjah Border</td>
<td>43.4</td>
<td>63.5</td>
<td>46.3</td>
</tr>
<tr>
<td>4</td>
<td>Jumeirah Road to Sharjah Border</td>
<td>Sharjah Border to Jumeirah Road</td>
<td>16.2</td>
<td>116.4</td>
<td>618.5</td>
</tr>
<tr>
<td>5</td>
<td>Bur Dubai to Burj Al Arab</td>
<td>Burj Al Arab to Bur Dubai</td>
<td>39.8</td>
<td>82.5</td>
<td>107.3</td>
</tr>
<tr>
<td>5</td>
<td>Burj Al Arab to Bur Dubai</td>
<td>Burj Al Arab to Bur Dubai</td>
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<td>58.5</td>
<td>32.1</td>
</tr>
<tr>
<td>6</td>
<td>Abu Dhabi Border to Rashidiya</td>
<td>Rashidiya to Abu Dhabi Border</td>
<td>35.9</td>
<td>60.5</td>
<td>68.5</td>
</tr>
<tr>
<td>6</td>
<td>Rashidiya to Abu Dhabi Border</td>
<td>Abu Dhabi Border to Rashidiya</td>
<td>39.9</td>
<td>49.4</td>
<td>23.8</td>
</tr>
</tbody>
</table>

Travel time comparison between 2004 and 2017 clearly shows that travel time has increased on all routes. Travel time increase ranges from just under 24%, on route 6 in the direction of Abu Dhabi border, to almost 620%, on route 4 in the direction of Sharjah border. The significantly high increase in travel time on route 4 is due to lack of adequate road improvement on Al Ittihad Road/Al Wahda Street at the Dubai and Sharjah border to match the increase in daily trips resulting from population growth and increased activities between the two Emirates over this period. Figure 5 shows travel time comparison on surveyed routes between 2004 and 2017.
Since the formation of RTA late 2005, the Dubai’s transportation network has significantly been improved and expanded to include new roads, metro lines, tram line, bus routes, water transport, taxis, etc. However, with all these investment and improvements the level of urban mobility has not been maintained since 2004 based on limited travel time surveys.

Travel time comparison shows that the minimum increase in travel time is about 10 minutes on route 6 in Abu Dhabi direction. Taking the 10 minutes as indication of minimum increase in travel time on most routes in Dubai, since 2004, then the annual time and money loss to travelers and the whole of Dubai economy would be significantly high.

Clearly, the pace of development has outstripped transport infrastructure provision in Dubai resulting in lower mobility. There is therefore a need to supplement the existing approach in developing transport plans through development of city-wide integrated land use and transport plan, priority for public transport, expansion of mass transit lines and development of traffic operational tools for managing existing traffic jams.

5 Managing Mobility

In a fast developing city, such as Dubai, there is significant and continuous pressure on transport authorities to provide adequate and appropriate infrastructure to cope with both expansion of the urban area and increase in number of daily trips. However, planners and engineers require reasonable amount of time and necessary tools to develop appropriate transport plans that not only could cope with short term traffic issues, for maintaining an acceptable level of urban mobility, but also to cater for long term travel demand.

However, in the Middle East, the planners tend to focus more on long term demand forecasting using advanced multi-modal transport models. Although, it is important and necessary to forecast long term demand it is also essential and probably more pressing to develop short term plans and focus on addressing current traffic issues in order to maintain acceptable level of mobility.
Dubai is a unique and dynamic city with many mega developments that take many years to implement and in most cases go through several updates before completion. For example, Palm Jumeirah was planned some 20 years ago and it is still not fully completed. Furthermore, Dubai attracts many local and international investors that do not necessarily occupy the property they invest in. In such a dynamic and changing environment it would be difficult to produce accurate or fairly reliable demand forecasts. Yet, significant efforts and investment by the authorities are put in developing advanced demand forecasting models. Traditionally, these models are developed to forecast travel demand for a given land use scenario and test alternative transport plans with a view of selecting the most optimum plan for different horizon years. These models are typically used for medium to long term horizons.

Demand forecasting models include the traditional four stages of trip generation, trip distribution, modal split and trip assignment. However, details within each stage are structured with a view of representing local conditions and reproducing the travel behavior of various population groups for different journey purposes. A key feature of the model is the relationship between the base conditions and future projections. However, in developing countries and in particular in the Middle East, the patterns of land use and daily trips are continuously changing and, in most cases, completely different between the base and future years. This is the main reason that the existing models in Middle East are absolute rather than incremental which means there is no direct relationship between base and future year models, other than the model parameters. This would therefore cast doubt on the reliability and validity of forecast year projections. Given the extensive efforts in validation of the base year model, it would be better to develop an approach that establishes a relationship between the base and future year values with a view of increasing the reliability of future year model output. This would, in effect, be combining the incremental and absolute modelling approaches.

In Middle East, even with the most advanced software and modelling techniques it would not be possible to produce reliable travel demand forecasts in line with international standards mainly due to continuous change in population, employment, land use development and travel pattern. Nevertheless, it is essential an acceptable level of urban mobility is maintained. This would require traffic operational models with on-line traffic monitoring capability to predict short term traffic levels on key corridors and roads and develop appropriate mitigation measures to manage incidents.

6 Remarks

In Dubai, the fast pace of development has affected the level of urban mobility despite significant investment in transport infrastructure. Dubai is a unique city with continuous development growth and diverse population of different income levels and travel needs. Furthermore, significant level of daily trips from the neighboring Emirates use Dubai’s transport infrastructure. The concerned authorities therefore are faced with huge challenge of maintaining an acceptable level of urban mobility.

Examples from the developed cities show that a comprehensive package of transport plans and measures are required to cater for travel needs of all segments of population and maintain acceptable level of urban mobility. The components of the transport package may vary from one city to another depending on local population characteristics, land use pattern and travel conditions but it generally includes development of an efficient and integrated land use and transport plan, balance between pace and density of development growth and transport infrastructure provision, appropriate transport policies, provision and priority of city-wide public transport network, appropriate modelling tools including demand forecasting and traffic operational models and application of advanced technologies to make efficient use of available transport infrastructure.

It is fair to say that over the last decades Dubai has invested significant efforts and investment in developing an efficient transport infrastructure to cope with the increasing travel demand. However, additional plans and measures including development of a city-wide integrated land use and transport plan, expansion and prioritization of public transport network and development of traffic operational model would contribute to maintaining acceptable level of urban mobility.
Reference:

1. Strategic Planning Department, Roads and Transport Authority of Dubai
ABSTRACT:
Dubai is one of the fastest growing cities in the world on both economic and tourism platforms, and it is said to have one of the most advanced public transportation systems in the world. The Roads and Transport Authority (RTA), formed in 2005, is responsible for planning and meeting the requirements for all things transportation in Dubai and between Dubai and other Emirates. Presently, Dubai is ranked the most congested city in the Middle East and with such ranking there needs to be certain adjustments made to compensate for the growing population. In this study, the Dubai public transportation system is put under the microscope and questions arise as to how efficient it is in solving the congestion problem in Dubai at peak hours. The operation system for the metro is evaluated from data collected during rush hour conditions and the service rate was compared to the high in demand service on major roads in Dubai during peak hours. One of the main propulsion for this research was the nightmarish traffic condition for the infamous Dubai-Sharjah commute. After analysis of the existing system, alternative designs were explored and case studies were used to evaluate the estimated improvement if actually implemented. The research and adjustments were made with the economic and environmental aspects kept in close check.
1 INTRODUCTION

Dubai is one of the fastest growing cities in the world on both economic and tourism platforms, and it is said to have one of the most advanced public transportation systems in the world. The Roads and Transport Authority (RTA), formed in 2005, is responsible for planning and meeting the requirements for all things transportation in Dubai and between Dubai and other Emirates. Presently, Dubai is ranked the most congested city in the Middle east and with such ranking there needs to be certain adjustments made to compensate for the growing population. The public transportation of Dubai currently consists of the taxi, tram, ferry, abra, bus and metro system. The Metro system consists of two lines the red line and the green line; with the red line consisting of 29 stations, running from Rashidiya to UAE Exchange and the green line consisting of 20 stations, running from Creek to Etisalat. The metro was opened on the 9th September 2009 with only the red line fully effective. The green line was officially opened on the 9th September 2011 (2). RTA estimates more lines being added by the year 2030. Given Dubai's recent up rise in the last 30 years, the transport system is phenomenal when compared to other countries with older historical presence. However, the rapid increase in expats and immigrants in the city has put a strain on the service of the public transport system to the society.

1.1 Problem Statement

With the growing increase in population, the Dubai roads have come overcrowded during peak hours, which range from 6:30-8:30am and 5:30-9pm. According to an article written by Vicky Kapur in Emirate24/7, the number of registered cars in Dubai has increased from 0.76 million in 2006 to an alarming 1.4 million in a city populated with 2.35 million people as of 2016 (3). With these estimations, it suffices to say that in every two Dubai residents, one has a car. Kapur explains that with these assessments, the number of vehicles on Dubai roads is 540 for every 1000 residents which is more than the two metropolitan cities; London and New York. Various factors contribute to this clog in the traffic flow on multiple roads at peak times; however, the main focus is on how the public transport system can be used to improve the present situation and can be optimized to its maximum capacity. The public system is set to expand its capacity in the next few years, but is this an expansion in the right direction? As stated earlier RTA announced a future addition of 197 stations branching into more areas in the city, this expansion though feasible does not pull the majority to the use of the public transportation sector. The present design of the metro can be modified to appeal to the population, thus reducing the vehicular congestion at peak hours. Also, alternative designs can be implemented to reduce the high flow at set hours.

1.2 Objective

The main objective of this project is to analyze the traffic flow and congestion on Dubai roads at the various peak times and find ways to reduce this vehicular traffic by derailing the traffic towards the public transportation sector. The metro system was specifically analyzed; however, some modification was suggested for the other modes of public transportation. Most of the research done was based on weekday peak hour analysis.

2 EXISTING CONDITIONS AND CONSTRANINS

The intense increase in cars has greatly increased the congestion of vehicles on the highways during peak hours. This congestion can also be attributed to the large number of traffic towards and away from the nearby Emirate, Sharjah during rush hour. This increase of flow in this direction could be because of the low rent and amenities in the adjacent Emirate. They are various areas in Dubai affected by the congestion of vehicles at rush hour; however, they are
other factors that affect the flow of traffic. Some include bottlenecks, lack of exits, poor traffic conditions and narrow roads.

3 METHODOLOGIES

3.1 Data Collected for Metro System on Weekdays

The Dubai metro runs 7 days a week with the average start time at 5:30 am and closing at 12pm every weekday. The frequency of the train arrival is heavily dependent on the time of day; however, on average there is a train every 5 minutes and at rush hour the time interval drops to 2-3-minute intervals. Data was collected on multiple days of the week and the average for each day at each station were recorded and put in Tables 1 depending on what time the data was taken and which direction was being monitored.

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3.2 Data Analysis

The values collected were run on excel and a detailed graph can be seen in Figure 1.
3.3 Discussion

As seen in the data collected in Table 1 and later plotted graph, the number of people who use the metro at both morning and evening peak hours varying per day and also in direction. The flow of people towards the Rashidiya direction on weekdays from 6-8am is less than that in the UAE Exchange direction likewise the flow of people towards the UAE Exchange direction on weekdays from 5-7pm is less than that in the Rashidiya direction. The reason for this may be because of accommodation pricing and location of the business district. The majority of workers work in the roaring Business Bay and Financial Centre areas, while living in the Southern Union and Burjuman area as a result of cheaper rent. So at the beginning of the day, the flow towards the business district is high which is noted in Figure 1 and the flow towards the Rashidiya district at the end of the day is high which is also noted in the figure. The flow of passengers in the counter-direction of the expected or estimated flow was also recorded and the massive decrease in numbers during both morning and evening rush hour was noticed.

The overall number of passengers in the past three years can be seen in the appendix. Looking at the data, the increase in passengers at each station over the years can be noticed and gives evidence of the fast-growing population of Dubai. As the train commuters increase, likewise the road commuters also increase. The congestion on Dubai roads has grown to alarming heights in recent times. The Sheikh Zayed road, original capacity of 7,000 vehicles per hour, is almost reached during the peak hour traffic and the service of the road at that particular moment reduced. The congestion on most roads coupled with the congestion in the metro makes for an insufferable transport system. This needs to be remedied and in the next sections, plausible options will be analyzed.

4 SMART SOLUTIONS

After analyzing the current service of the road and the metro system from the data collected and researched on, it is more evident than ever that some improvements can certainly be made to compensate for the recent increase in population. The metro system was the only mode of the public transport sector monitored during the course of research
for this project; however, modifications can be made to other modes of public transport to resolve the congestion problem. Listed and detailed below are a few solutions that can be implemented alongside case studies of countries that have already implemented these methods and seen positive outcomes.

4.1 Alternative 1

As mentioned in the discussion, the high congestion presently in metro stations in the business and tourist districts at peak hour is appallingly high and this reduces the service of the metro at that time frame. The metro stations are built in accordance with a permanent design, so reconstructing the station to increase the number of cabins would be uneconomical and insensible. To maximize the metro during rush hour, a different type of train should be used. Instead of the typical type with multiple seating, as seen in Figure 2, a light railway train with little to no seating should be used in order to fully maximize the carrying capacity of the metro during that time frame. If trains with such design are used only for peak hour transportation the number of people transported during that time frame will greatly increase all the while reducing congestion and increasing the carry capacity of the train per trip.

However, not all the seat will be removed as there will be special cases in which seating is prioritized; for example, pregnant women, disabled people and the elderly. So it is reasonable to leave a few seating positions for these groups of people. The newly designed cabins should have multiple railings in each cabin so as to prevent slippage when the train is in motion. This new design is similar to that of the aircraft-to-terminal shuttle which is used to transport passengers from the aircraft to the terminal in cases where the aircraft lands far off from the airport terminal. In cases like this, the topmost priority is to carry the highest number of people at a time. An example is shown in Figure 3. These vessels are actually made to carry more people and luggage rather than built for comfort. If this is implemented in the Dubai metro specifically for peak hour conditions, there will be a huge improvement in the metro congestion.

![Figure 2: Interior of a standard class cabin in the Dubai metro.](image)

![Figure 3: Interior of a typical aircraft to terminal shuttle.](image)
4.2 Alternative 2

Another system that can be implemented is the Bus Rapid Transit (BRT). The BRT is an option that relies on the use of segregated lanes for buses to ensure faster and more efficient bus travel. This segregation ensures the public bus does not mix with the normal flow of traffic thus during peak hours this ensures that buses are not disrupted by the traffic congestion. The lanes are usually at the side of the road or in the center and they are constantly monitored to ensure normal vehicles don’t use these lanes. Implementing the BRT system in Dubai would greatly reduce the number of passenger vehicles on the major roads. The increase in service of the bus system will certainly lead to an increase in the flow of user to the public system. So, if people can use the bus and be ensured no delay whatsoever, there will be fewer passenger cars on the roads.

Figure 4 shows the possible outcome for the nearest future. A simple yet explicit cycle illustrating the domino effect the rise in population will have on the transportation sector. The BRT system will not only decongest the roads but will also have an environmentally friendly impact on the city. The lesser the cars are, the fresher the air is. The number of passengers that use the bus has significantly increased over the past three years however research shows that if the BRT is applied to Dubai roads there will be a much more tremendous increase. According to an evaluation of Bus Rapid Transit (BRT) in Dubai:

"A proactive and attractive bus transport has the ability to reduce car population, decongest the city roads, reduce toxic emission, protect the urban environment and combat global warming. There is an imminent need to increase the attractiveness of bus transport by reducing the journey times in order to encourage a substantial shift from private cars. Public transport’s appeal hinges on its speed and regularity. There are research works to prove that public transport share is directly influenced by the ratio of public transport average speed to automobile average speed. The most popular public transport networks are the ones that offer the best speed and regularity compared to the car. At present, bus journeys take twice as much time as cars to traverse similar trips" (5)

The BRT has been implemented in various countries and has yielded positive outcomes for most of them.

Figure 4: Effect of an increasing population.
4.2.1 Case Study: Lagos, Nigeria

Lagos is located in the south-western part of Nigeria, and it is known as the most populated city in Africa, it houses both the extremely poor and the overly rich. It is known to be the hub of Nigeria and therefore it is house to many people. The Bus Rapid Transit (BRT) was commissioned on March 17, 2008, by then governor, His Excellency Babatunde Raji Fashola. The lane ran for as long as 22 km spanning from downtown Ikorodu to Funsho Williams Avenue. A section of the BRT lane in Lagos can be seen in Figure 5. Maintaining the use of the lane was hectic as Nigeria is a country of great corruption and uncooperative citizens, but this was overcome by implementing soldiers at certain points to prevent normal passenger cars from using the designated lanes. Since its commencement the number of passengers increased to a roaring amount of 200,000 passengers daily and 4 million since its opening. Employment opportunities were made available and the traffic congestion reduced considerable (5). The Lagos BRT system caters to a city of over 21 million people and it does so effectively even though it is situated in a third-world country that lacks most resources. The BRT had an impeccable impact on the congestion problem in Lagos so it can similarly relieve the congestion in Dubai.

4.3 Alternative 3

Dubai is considered an oasis in the desert for tourists and expat from all over. It strives on being a place of luxury and high expense and the people have high-end lifestyles. Knowing this, it is hard to assume or estimate that the high-class population would be willing to slum it in the typical concept of public transportation just to get home faster. Therefore, a different approach needs to be looked at all the while it being effective in reducing the congestion on Dubai major roads. The Personal Rapid Transit system (PRT), all be it expensive, would help a great deal in reducing the population of the middle class and upper-class drivers on the roads during the peak hours. The PRT can be situated in areas that are currently undergoing high urban sprawl which include the Dubai Silicon Oasis, the Dubai Sports city, the Dubai Motor city and any other areas. There has been a sharp increase in the number of housing being built over the past three years. A satellite image of these locations experiencing urban sprawl in Dubai between 2009-2015 can be seen in Figure 6. According to expat arrivals, a great number of these housing facilities are only accessible by the middle and upper middle class in Dubai (7). This is the class the PRT should target, the working class which wishes to experience luxury all the while avoiding the traffic on their way to work.
The PRT system consists of small pods that are self-automated and run on special tracks that can span for kilometers. They can be bigger in size but in that case, that system would be called a Group Rapid Transit (GRT). These specialized taxis although expensive can do wonders in decongesting Dubai at peak times. If stations were implemented at urban areas and run through the traffic straight into the hub of business in Dubai like Business Bay, Media city, DIFC etc. The likelihood of the high-end class making use of it is high. The PRT is still in its beginning phases but it has been implemented in a handful of places in the world (8).

4.3.1 Case Study: Heathrow Terminal 5, London, United Kingdom

The PRT operating in the London Heathrow airport spans over 3.6km and consists of 18 driverless vehicles which run on low energy. Ever since it opened consumers have complimented it for its easy accessibility, comfort, convenience, and safety. The pods do not waste too much time at stations and its speed, though slow allows the passengers to have a bump-free journey in good time. It is sustainable and it has helped reduce London's carbon print. Similarly, the PRT in Masdar city, Abu Dhabi has shown tremendous result since opening in 2014, with 300 pods, 85 stations thus making on average 150,000 trips a day. Pods departing from the Heathrow terminal can be seen in Figure 7. The PRT system has some minor flaws in the sense that it is expensive to set up the tracks and buy the pod but in the long run, it is a huge bonus in reducing the traffic on Dubai roads coming from residential areas to the business districts.

Although the options explored seemed to tackle the issue of congestion in Dubai, other factors seem to have been overlooked such as the change in climate condition in Dubai. One of the methods used to make the public transport sector appealing to all classes was to make it convenient. Dubai is prone to heat and humidity in the summer months. During this time frame, people are less likely to use public transport as it would be strenuous and tiring. At this point in time, access to any mode of public transportation should be made easy and promotions should be implemented to cajole the users to engage in the use of public transport even when under such excruciating conditions.

The drainage system in Dubai should be considered. Dubai does not have a good drainage system, so when unexpected rain falls the service of the roads gets worst thus affecting all modes of transportation. New innovative bus...
to boat vehicles can be added to the public sector to compensate in situations like this. An example of such can be seen in Figure 8. Other options, could involve changing a few transport policies such as the time heavy duty vehicles can be on certain roads, also more fees and rules could be enforced on passenger car holders to pressure them into using the public system sector thus gradually reducing the traffic congestion.

Figure 8: Bus to boat type of bus.

The world is fast evolving and for societies to keep up with the dynamic changes going on new innovations must be applied. Smart transport solutions deal with the utilization of innovation to improve vehicular availability/accessibility and the transportation sector for convenient movement in the general public all the while conforming to modern approaches. The solutions given below are additional plausible solutions for the congestion on Dubai streets.

- Electronic payment option should be implemented in the RTA taxis. RTA needs to adopt a more advance payment strategy. Users should be given the option of paying using an online virtual Identity which entails all their financial details. UBER is growing constantly in the UAE demographic and this growth is poaching consumers from the RTA public transport. An update from the archaic and normal means of taxi transportation would both increase revenue and reduce private vehicles on the road and this will create an uphill effect of reduced carbon emission.
- Bus routes should be recalibrated and reconstructed to mitigate harmful and wasteful emission.
- Implementing seaplane service would be able to maximize all routes of public transport. Dubai has various areas with access to a body of water and applying this new and innovative form of transportation increases Dubai’s’ level of advancement to remarkable heights. This transport mode was used in the past to move between the Emirates, so routes can be easily calibrated.

These solutions will reduce the number of personal cars all the while increasing the efficiency of modes of public transport in the Emirate.

5 CONCLUSION

The Purpose of this project was to evaluate the congestion of traffic in Dubai during rush hour and find solutions using the public transportation sector to reduce the congestion. The solutions suggested were environmentally friendly and economically plausible, meaning it would yield profit in due time after implementation. The first alternative of different trains being used during rush hour may pose a great adjustment problem because it would take a while for the consumers to get used to such change. Economically this would profit the government because more people get to use the metro at peak times thus increasing the profit margin for the metro; however, the revenue on vehicles and vehicle-related expenses would drop a get deal and this would also affect the economic aspect of things. These factors are applicable for all alternatives suggested. They all help the environment because less congestion means less emission; they all benefit the societal aspects of things by increasing the tourist flow to Dubai and they all benefit the economic aspects of things for the public transport sector all the while reducing the revenue gotten from private car owners. The main obligation for any of the designs is to the public. The consumers’ needs should be the topmost priority and until most of their needs are met different suggestions must be made.
REFERENCES


# APPENDIX

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*The Red Line was opened on September 2009

Source: Roads and Transport Authority

**Figure A: Metro Passengers' Trips by Station - Red Line***
1: Summary

Automatic parking systems (APS) are electro-mechanical systems where cars are parked in terminals by drivers; the system then parks, and later retrieves the cars. With APS more spaces can be built on a plot than with multi-story. If the cost of land and/or the value of extra-parking; is high, this can sometimes make sense. After Japan’s Parking Law was enacted, a few million low-throughput automatic-spaces were built in Asia; in garages where cars are used infrequently. Worldwide only about twenty large-scale systems with throughput-capacity sufficient to handle offices/retail; have been built; four are in Dubai. We evaluated the systems in Dubai in 2015. No major safety-incidents or problems with reliability were reported to us; either by users or operators. One did not have capacity to handle peak-throughput during rush-hour if the garage was more than half-full. At this location many office-workers preferred basement parking, which was an option. Computer-logs for another system with sufficient throughput capacity, showed retrieval averaged 3.3 minutes with 99.5% in less than ten-minutes; this was acceptable to users who said they preferred the convenience and comfort of automatic to the multi-story option. The main reason automatic was chosen for the four garages, was because after accounting for land-value, and the time cost of money, the automatic option cost per space worked-out at 25% to 50% of what other options cost; and it was also was perceived to provide a much better quality parking-experience, so long as design throughput-capacity was sufficient to handle peaks.

2: History of Automatic Parking

It wasn’t until Japan’s 1957 Parking Law, which banned on-street overnight-parking, was enacted in 1962¹; that the idea of automatic parking systems took-off. Over the next twenty years a few million spaces were built in Japan, Korea and more recently, in China. Those systems typically lack redundancy, so when they break down (infrequently), all the parked cars are stuck; also if everyone wants their car at the same time, the last one in the queue might wait half-an-hour. But most car-owners using these systems don’t drive every day so throughput capacity is not critical. The popularity of the systems in Japan is in part because in many cities you can’t buy a car unless you can prove, you have somewhere to park it, also land costs are high; so the smaller footplate of automatic parking compared to multi-story, means the concept can often be economically viable.

Fig 1: Low-throughput Automatic Parking Systems

*Source: Internet Search*
Advances in PLC technology in the 1990’s meant automatic parking systems could be built to handle peak-traffic-flow required for offices, retail and park-&-ride; where typically there needs to be capacity to empty a full garage, or fill an empty-one, in less than three hours. The new systems also had high redundancy, so in case of any malfunction, only one a few cars may gets stuck.

The main reason for considering automatic parking is that it is possible to fit two-to-three times more cars into a given space compared to conventional multi-story. This means it’s sometimes possible to build more functional real-estate, or in the case of park-&-ride, persuade more people to use public transport. In these situations, the cost-premium per-space when compared to conventional multi-story car-parks; can sometimes be more-than covered by the increase in value of the development. Another reason is that users can park and retrieve their cars right in front of the entrance to the building, rather than navigating a sometimes inhospitable car-park, this can also add value; particularly in very-hot or very-cold countries where walking outdoors is a challenge; or for high value developments, where everyone wants a reserved parking space right next to the front-door; and they are willing to pay-extra for that convenience. One other advantage is that car doors can be opened fully inside the parking-stall, which adds to the comfort of parking.

So-far, the idea of high-throughput automatic parking never really took-off; we identified twenty-three automatic garages world-wide that have more than 300-spaces and can nominally be considered high-throughput. Only seven had capacity to empty or fill in less than three hours, which is normally required for office developments.

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Table 1: High-Throughput Automatic Parking Systems with more than 300-spaces installed world-wide up to 2016.

Sources: Information provided by suppliers directly and from web-sites.
The number of installed spaces has been increasing steadily since the first system was built in U.S.A. in 2002; but the concept shows no sign of taking off, and only one-to-two new units are built per-year.

![Graph showing the development of high-throughput automatic parking with more than 300 spaces](image)

**Fig-2: Development of High-Throughput Automatic Parking with more than 300-spaces**  
*Source: Table-1*

The time from inception to realization of the concept is long, because the systems typically serve large buildings that take time to plan, finance, and build. For example the system installed in the Conrad Tower in Dubai was planned between 2005 and 2007, but the building opened in 2013. The drop in growth of new systems in 2011, in many countries, was likely due to the Financial Crisis in 2008/9.

### 3: Background

From 2003 to 2007 ABMC was part of a team planning two mixed-use buildings in Dubai where automatic parking was used to provide some of the required parking capacity. To evaluate feasibility for the concept, we visited installations in U.S.A., Austria, Turkey and Korea; we spoke with suppliers; and we hired an automation consulting company to prepare technical specifications. We also researched capacity and throughput requirements by observation at similar buildings to the ones that were being planned, so as to provide a basis for decisions on optimal parking-ratios and throughput capacity; and we did market research of office tenants in the area to provide a basis for estimating the value of parking and parking attributes.

The decisions to go with automatic parking on both projects were because this allowed much more high-quality office-space (i.e. with ample parking), to be built on the sites. At the time, prevailing expert-opinion was that there would be red-hot demand for offices in Dubai in the future, this was reflected in prices achieved for off-plan office-space in the Dubai International Financial Center (DIFC); and so it was reasoned that extra offices enabled by extra parking capacity, represented the highest-&-best use.

In the event many other developers in Dubai also decided that there would be a red-hot-demand for offices, so there was over-building; then demand was affected by the Financial Crisis; so today neither of the offices are full; and so the value-add achieved by automatic parking on these developments is not clear.

The buildings were the Conrad Tower and the Buildings-by-Daman; both had about 400,000 sq/ft of offices with parking ratio of about one-space per 40-m2 of GFA, with potential for over-spill into basement hotel-spaces during the day at the Conrad, and to multi-story residential/hotel-spaces at the Buildings-by-Daman, translating to an effective parking ratio of 1:30.
This was a lower ratio than the Dubai Municipality code (1:60) and the DIFC code (1:40), the reason for that was because our market research showed low ratios achieved better rents (when offices were full). Specified throughput-capacity at peak was sufficient to empty a full car-park or fill an empty one in less-than 2.5 hours, i.e. roughly 400 Vehicles per hour per 1,000 spaces. Both buildings were completed in 2013.

The system in the Conrad was supplied by Westfalia, an Austrian/German company that specializes in automatic warehousing, and the system in Buildings by Daman was supplied by Sotefin, a Swiss-Italian company which is the largest supplier of high-throughput automatic parking in the world. Subsequent to the decision to install automatic in the garages we were involved in, Robotic Parking Inc (a U.S. company), was awarded contracts to supply a 765-space system at Ibn Batuta Offices (opened in 2009), and 1,200 spaces at Emirates Financial Towers (EFT) in DIFC, (opened in 2011), at the time the EFT system was the biggest automatic-car-park in the world; measured by spaces, but not by throughput capacity.

In 2015 we were asked to look at the feasibility of using automatic parking for park-&-ride for a metro system in Saudi Arabia; this gave us an opportunity to revisit the projects we had worked on.

As part of this study, the four automatic-car-parks operating in Dubai were evaluated. The reason automatic parking was contemplated for the metro was mainly lack of availability of land for parking close to metro stations. The planners thought parking was important, because in Saudi Arabia people typically drive even short distances rather than walking; because it is often hot. It was reasoned that by having parking at metro stations, that might increase utilization of the metro and therefore help achieve one of the main objectives of the project which was to take cars off roads during peak-driving times.

### 4: Methods

We visited each of the four parking garages and interviewed attendants, technicians/operators, managers, owners-representatives, and suppliers (total thirty interviews). We conducted open-ended intercept interviews with users waiting for their cars (total 120 interviews). We were provided access to observe operation inside all the garages along with technicians operating the systems, and in some cases, sight of operations and maintenance manuals, planned-maintenance procedures and records. For one garage we were provided with computer-logs for operations in the month of August 2014, which we analyzed. We conducted observation studies of throughput and we estimated the ratio of utilization of automatic-parking, at peak, to utilization of conventional multi-story or basement parking, which was an option in all of the garages.

### 5: Architecture

All of the systems in Dubai are “lift-and-run”, the car is parked in a “transfer-terminal” (gate), and once the driver and passengers leave the terminal, a garage-door is closed (preventing people from being in the transfer terminal when the machine starts to move the car), and the car is transported into a lift to take it to a storage level, it is then transported out of the lift into a “transfer-alley”, and parked. For retrieval the process is reversed; most of the systems turn the car around at some point so that the car can drive out forwards from the same transfer-terminal or “exit/entry gate” (EEG), where it was parked (or one close-by); in one system (at the Conrad), separate entry and exit transfer terminals were provided and cars were not turned-around by the system.

Commonly there are two rows of parking stalls on either side of the transfer-alley, so when the garage is full, to retrieve a car from the second row requires that the car in front is moved out of the way; this does not unduly delay retrieval so long as there are two transfer vehicles on each level, which is in any-case required for redundancy. In some systems it was possible to order the car via an SMS message, so during the time the user is navigating the lifts in the building, the car is being retrieved, so waiting-times are minimized. Digital photographs are taken of cars parking, so if the user loses the parking ticket, but they remember approximately at what time they parked, they can find their car.
5.01: Pallets or Buggies
In three of the systems in Dubai cars are parked on steel pallets; these are transported through the parking garage with the cars riding on top. In the one system supplied by Sotefin the cars are transported on a “buggy”; the car is driven over the buggy in the transfer-terminal; then after the driver and passengers leave the transfer-station, the car is aligned by a rubber-covered bars that push the tires to align the car with the buggy, which then picks it up by the tires using a comb-system, and transports it into the lift. Another buggy picks the car out of the lift and transfers it to a transporter which brings the car to a parking-slot, where the buggy parks the car.

Advantages/disadvantages of pallets/buggies:
- Both systems have sensors to work out where the car is during parking, these are linked to displays that say “go-left/right” and “go forwards/back”. The tolerances required for parking is lower for pallet-systems than buggy-systems which are nominally easier to use, although most people could park successfully first-time with both systems, without coaching.
- In all systems the users are told to engage the hand-break via a sign and sometimes a recorded voice, the reason for this is because the car risks rolling-off the transporter when the buggy/pallet is accelerating or
de-accelerating, this appears to be more important for pallet-type systems than buggy-systems. This is actually not correct: looking at the wedged arm on the buggy system — it is not more wedged as the robotic pallet groove that acts in exactly the same way as roll-off barrier. Moreover, if in a buggy system the driver does not engage gear or brake, then the car is unsecured in the parking space location at rest.

- In one system (the Conrad), pallets were flat (the other pallets had channels plus roll-off barriers for tires), and in this system attendants told us that it was fairly common (about one incident a week) for parking-cycles to be aborted when the user did not apply the hand-break. This triggered an alarm and stopped the parking process; which could only be re-started after the parking attendant manually opened the parking terminal so the driver could get back in the car and put on the hand-break. We received no reports of issues arising from hand-breaks for the other systems or from buggy-systems.

- A criticism of the buggy-system is that potentially the buggy could damage the under-side of cars with low clearance. We heard that this was an issue with the earlier versions; however Sotefin, the market leader in these systems, introduced a new model in 2002 which they said solved this problem. We received no reports of any problems of damage to cars in the Sotefin garage we inspected.

- There is a limit to the weight of the car that can be handled by the buggy-system of about 2,700 kilos; some cars in Dubai weigh more than that (for example a Nissan Patrol with a full tank), there is theoretically no limit for pallet systems, although the systems we inspected had a nominal limit of three-tons, and analysis of performance showed heavier vehicles took more time to retrieve.

- The garage structure for pallet-type systems is normally steel-frame, for buggy-systems it is typically concrete, so the inside resembles a normal car-park except there are lifts rather than ramps between floors. This means that the interior of the buggy-style car-park is safer than for pallet systems; and theoretically drivers can walk unattended to pick something up from their parked car, rather than retrieve it and then re-park it; although in the buggy-system in Dubai this was not allowed, even for people accompanied by an attendant.

- There is vibration in the steel frames of pallet-type systems as cars are moved around, this means that fire-protection is not possible because the retardant material falls-off. So if the space above the car park is inhabited, it’s necessary to support it with independent concrete columns in order to achieve A-60; so effectively there are two structures, which is more expensive and less efficient in terms of space-saving. Incorrect / we have designs for robotic garages with concrete support

- We heard of one fatal accident inside a steel-frame automatic car-park (in USA), caused by vibration loosening bolts, causing a part of the structure to collapse, killing a maintenance technician. This eventuality can be mitigated by specifying self-locking bolts, and regular inspection with a torque-wrench, as part of the planned maintenance. Please correct: the attached article states the technician fell from a lift / not from loosening bolts of a steel structure

- It should be added that a palletsystem prevents any drippings from cars onto machinery and / or other vehicles; it also is a positive means to protect the vehicle from any interference from below or the sides. Thus, from a product liability point of view providing the best protection possible.

5.02: Space Saving
With lift-and-run the effective foot-plate of a parking-space is about 20-m2 including the transfer alley, compared to typically about 35-m2 in a normal multi-story car park, plus-or-minus depending on the size of bays, whether there is angled parking, whether the parking is in a basement or free-standing, and whether there is an express-lane to help achieve required throughput at peak times. The height of each level with automatic is about 2.5-meters compared to the typical slab-to-slab dimension of 3.8-meters with multi-story; so the volume of a space is about 50-m3 with automatic; compared to about 135-m3 with multi-story, and so about 2.7 times as many car-parking spaces can fit into a box using automatic-parking, than one using multi-story.
Another advantage, sometimes; is that there is no hard-limit to how many levels of automatic parking can be installed; each extra level adds only about 5-seconds to the retrieval time; and usually retrieval-time is not the main design constraint (throughput is).

Most developers agree that once a multi-story car park goes over five levels, either up or down, the customers start to complain, and so the unit-value of the extra spaces goes down. Low-cost buildings can have ten levels of multi-story, but they don’t get Grade-A rents. This can mean that “like-for-like” three-to-four-times more spaces, of equal or better quality, can sometimes be built on a given plot, using automatic, compared to multi-story.

The Conrad system has twelve levels above-grade and 1,000 spaces, the alternative in the same “box” with multi-story would have been seven levels with 600 spaces.

![Fig 4: Automatic Parking at Conrad Hotel & Offices in Dubai](image_url)

**Fig 4: Automatic Parking at Conrad Hotel & Offices in Dubai**

*Courtesy: Westfalia*

**6: Sizing**

Developers typically either build too much parking; so they waste their money because some of the parking is never used, or they build too little, in which case some of the money they spent on the building served by the parking is wasted, because tenants are not prepared to pay a premium if parking is inadequate. There are three overlapping considerations when sizing any car-park:

I. **How many parking-spaces?**

Design guidelines prepared in 2003 by Halcrow for the Dubai International Financial Center (DIFC); stipulated a parking ratio of one-space for 25-square-meters of Gross Floor Area (GFA) of offices. At the time the ratio specified by Dubai Municipality was 1:60. Later, the developers who had bought land in DIFC persuaded the authority to relax the standard to 1:40. Who was right is a question about how long is a
piece of string? Currently in Dubai; offices are only about 70% full so the ideal parking ratio is hard to assess, also the Dubai Metro has reduced the demand for parking at offices.

Many developers just build the minimum to comply with code; this can be a false-economy. In Dubai the decision often ends up in a debate about whether to build an extra level of basement-parking, which, because of a high water-table; costs double above-grade multi-story; i.e. roughly AED 150,000 per space ($40,000) compared to AED 70,000 ($20,000) above-grade.

II. How much throughput is required?

Standards for throughput requirements for car-parks were first formalized by parking-designers in U.S.A. in the 1950’s. Recommended capacity for entering or leaving in units of “vehicles per hour per space provided” (VPH/Space); varies depending on real estate serviced; a typical industry standard is:

- Hotels/Residential 0.3 VPH/Space Empty a full garage in 3.3 Hours
- Retail 0.5 VPH/Space Empty a full garage in 2.0 Hours
- Offices 0.6 VPH/Space Empty a full garage in 1.7 Hours

During the planning of the Conrad Tower a study was done of cars arriving-at and leaving parking at nearby offices:

![Fig 3: Traffic Study API and Fairmont Office Parking](Source ABMC)

There were two surprises, first there was a high level of consistency in traffic flow regardless of the day of the week. Second, “rush-hour” both IN and OUT was spread over two-to-three hours. The reason for this was likely because the offices were multi-tenant, and also because in Dubai some companies deal mainly with Asia, so they get up early, some deal mainly locally, and some deal mainly with Europe and USA so they come into work, and leave work, later.

It is likely that the throughput standards for offices developed in U.S.A. in the 1950’s, were for a time when workers typically arrived at the same time and left at the same time, as still happens in many government buildings. For the Conrad Offices a peak VPH/Space of 0.4 was chosen as a basis for design (fill empty car park or empty full one in 2.5 Hours), i.e. 35% lower than the industry standard; this had a major cost impact.

Later, once it was known that there would be a metro station in front of the building, and also when it started to become clear that there might be over-building of offices in Dubai; design VPH/Space was reduced to 0.3, i.e. half the industry standard (above). It is currently hard to tell if the decision was correct because the offices served by the parking are still not full.
The decision on how much throughput to design for has major cost implications. With conventional multi-story this relates to whether to have one ramp, or two; and whether to have express-lanes or not; and if so, how far between express-lanes? With automatic this affects the numbers of lifts, and the numbers of transfer-terminals (exit/entry gates).

Another issue with throughput capacity is queuing when the car-park is filling up. Users typically don’t mind (much) waiting in line inside their car for ten minutes, when parking their car; but municipalities do mind if cars are queuing on public roads. Having sufficient gates with sufficient handling capacity is the key to minimizing the capacity for queuing on site.

III. Retrieval Time

The cost of situations where car-parks choke during peak times is difficult to evaluate. In some of the shopping centers in Dubai, the parking has a reputation for choking. Whether these would do better if the parking was better is debatable. Some residents of Dubai say they never visit some malls because of the traffic-jams in the parking.

Although it is quite complicated and requires special software, some parking consultants can model how long it will take to exit a parking-garage, the result of a model done for a car park with one ramp is illustrated below:

![Fig 4: Model Throughput for Multi-story Car-park with one Ramp](image)

In this case, if the building generated 0.5 VPH/Space at peak; if the target maximum driving-time to get a car out of the garage was say seven-minutes, then a one ramp multi-story should have no more than 500-spaces.

When we interviewed office-workers at Emirates Financial Towers, who chose to not use the automatic-parking; we were told that during rush hour sometimes it took twenty minutes for a car to be retrieved, and that was why they used the multi-story rather than the automatic. We estimated VPH/Space at 0.18, i.e. 30% of the industry standard and about half of the basis for design of the Conrad.

Based on observation we estimate that the automatic-parking in EFT was never more than half-full, people only used the automatic when they didn’t need their car during rush-hour. Lessons-learned; in Dubai, sometimes waiting twenty minutes for a car is not acceptable...although in Dubai it is common to wait more than twenty minutes for cars to be delivered by hotel valet-parking.

We were provided computer-logs for the automatic parking at Ibn Battuta Offices for the month of August 2014, during which time a maximum of 340 cars were parked in a day; the garage had a total capacity of
700 spaces, so it was never more than half-full. Average retrieval time over the month was 3.3 minutes with 99.5% less than seven minutes. 0.5% of retrievals suffered a technical malfunction (i.e. less than two-per day), in these cases the technician had to intervene and the retrieval took up to twenty-minutes.

Fig 5: Retrieval Times for Automatic-parking at Ibn Battuta Offices in Dubai

Data Courtesy Robotic Parking Inc: Analysis ABMC

This performance was acceptable to users who preferred to use the automatic-parking to a multi-story car-park under the main building. We estimated over 75% of office workers used the automatic parking; the main users of the multi-story were visitors.

The regular users we interviewed were quite sanguine about the occasional technical malfunctions in the parking system and in their minds the risk of this was outweighed the advantages of using the automatic parking, instead of the multi-story. Many had the mobile number of the lead technician on their phones, and they told us, “If there is a problem we just call Stanley”.

Other sizing considerations that affect design include:

- **Bay-size:** In Dubai the standard bay in a conventional car-park is 2.5 meters wide, this is often rather small in circumstances where a lot of users drive SUV’s; in higher quality developments bay-sizes of up to 3.0 meters are provided. The size of the transfer-terminals in the systems in Dubai ranged from 3.8-meters to 4.8 meters for pallet systems, and was 7.0 meters in the one buggy system which had a turn-table inside the gate. Often a constraint in the design of an automatic parking system is how many gates can be fitted in the available space.

- **Angled Parking:** American parking designers seem to prefer angled parking, perhaps because cars in America are typically larger than elsewhere, this makes parking easier, but takes more space. An advantage of automatic is that it’s easier to park and un-park, than in a conventional parking, because there is more space and there is a display saying “left a bit”...“right a bit”.

- **Walking distance:** Typically in Dubai it is common to limit walking distance inside a car park to eighty-meters; this is driven by cost because a major cost is elevators. With automatic parking typically the walk from where the car is parked to an air-conditioned space is less than thirty meters, this can add to the quality of the parking experience.

**7: Safety**

U.A.E. fire-code has a section on automatic parking based on the U.S. standard NFP 88A/ with the addition of wet-risers and smoke detectors.

We received no reports from users or operators of the systems in Dubai of any serious safety incidents. Trawling the internet and in conversations with suppliers; we found few reports of incidents worldwide. All of the systems in Dubai were attended at all times, typically by a minimum of two technicians with radios; regular users often had the mobile phone numbers of technicians and the number was displayed on signs. This level of attendance may have
contributed to the low-level of safety incidents, although it’s hard to be sure because at none of the installations was there a formal procedure for monitoring safety or documenting incidents and/or near-misses to the public, or to operators and/or maintenance technicians.

We were told by Sotefin that more than fifty smaller systems with less than 200-spaces; they have installed by them over the past twenty-years; are not attended, and as far as they reported to us, these have been broadly incident-free with regards to safety.

Insurance brokers told us it is cheaper to obtain Protection & Indemnity (P&I) insurance for an automatic car park than a conventional one, perhaps because in conventional car-parks there is a risk of collision and personal injury, including driving out of the side of a multi-story car park at height, which sadly occurred on one occasion in Abu Dhabi in 2015. In some countries also there is risk of mugging/rape in car parks which can increase premiums, although this is not generally perceived as a risk in Dubai.

Apart from the fatality of a technician (mentioned above), the following incidents were reported to us:

- Sotefin told us of an incident in 2005 in one of their systems where a woman pressed the accelerator instead of the brake and drove over the buggy into the lift, and was slightly injured. A similar incident happened in Abu Dhabi in 2016 in a multi-story garage, and the driver sadly died.
- We saw a 2012 press report of a user in the Robotic Parking Inc system in U.S.A. who was about to drive out of the transfer cubical, when the system closed the garage-door and started the process of removing the pallet, trapping the driver in the car. This was a frightening incident which damaged the car, caused by poor maintenance; there was no injury to the occupants of the car, although the fire-brigade had to cut an escape, because the doors and windows jammed.
- We heard of one incident at one of the systems in Dubai, where somehow the driver got trapped inside the transfer cubical after the garage-door closed unexpectedly, and ended up calling the police on his mobile phone to get out; because he didn’t know the number of the technician in attendance and there was no panic-button inside the transfer bay.
- We found two reports from an installation in U.S.A. where cars had been dropped from the top level onto the ground, during the first year of operation in 2002. This glitch appears to have been resolved and we did not hear of any reports of damage to cars in similar systems in Dubai.

All of the systems had sensors to detect movement inside the transfer-terminal, after the garage-door, or the access door; was closed, and the machine started to move the car; which by all reports worked-well. Technology exists to check for heart-beats inside cars (for example if a user forgot a sleeping child inside the car); this would however delay processing by 30-seconds, and was not installed at any of the garages.

In the event we heard of no reports of children being left behind. This would theoretically be more dangerous in a pallet-type system because of the risk of falling and electrocution inside the garage if the child woke up and decided to get out of the car. In the buggy-type system, the danger would likely be no different from the danger of a left-behind-child waking-up and wandering-around a normal car park. Certainly incorrect: even if there are concrete floors, there still are automatic machines running that will not detect the child / further, as stated above, the robotic pallet system can be installed as well easily in a concrete structure with decks All systems had internal CCTV, although this was not monitored constantly in any of the systems.

Some suggestions for improvements:

- Although suppliers and technicians were “safety-conscious”, we did not see any evidence of formal risk-analysis by qualified safety-professionals; or regular safety reviews and incident/near-miss reporting during operation; this is an area that could be improved in all systems.
Most of the systems had elaborate question-answer dialogues for users to participate in at the control consul; these were somewhat long-winded and irritating; and in some cases unclear or ambiguous; it’s likely that after a formal risk analysis these could be made clearer, and shorter.

Some suppliers told us that the reason for “over-complicated” dialogue at the consuls was, “legal”, although it’s debatable how much legal-protection these might provide in the event of a serious accident, and in any case this is un-tested. A better approach, particularly in U.A.E., might be for first-time users to be provided with a safety card to read and sign, with the technician trained to go through the contents to be sure the user understood all potential safety-risks.

In systems where there is a remote risk that a person might get locked in a transfer-terminal (as happened at least once), there should be a mechanism inside for the trapped person to open the garage-door from inside; along with a “panic-button”; and the telephone number of the attendant should be displayed.

Apart from the incident in U.S.A. where a car was caught in the pallet-retrieval system, we heard of no reports of the machines malfunctioning and endangering users and/or technicians. All-the same a procedure for stopping everything when the driver is in the car might be worth considering; for example by blowing the horn to activate an emergency shut-down, also technicians might carry radio “all-stop” devices.

8: Availability

The automation consultants appointed for the Conrad automatic-parking system recommended a target availability of 99.5%, as defined by DIN-VDI 3581; a German Specification for “Availability of Transport and Storage Systems”. We went along with this recommendation. The criteria for “availability”, was that the queuing time coming in should be less than five-minutes, and the retrieval time from initiation should be less than ten minutes. The supplier claimed a design availability of 99.65% for this definition; which is a higher availability threshold than that offered by the only other automatic-parking supplier that offers this quality-parameter, of 98%; although the meaning of availability is not specified.

One of the reasons Westfalia was chosen for the Conrad system was that they were able to commit to an availability target, possibly because the background of the company was automatic warehousing and this is a common quality benchmark in that industry; none of the other potential suppliers could commit to this without laborious caveats. In the other three systems, neither availability, nor throughput capacity, were clear deliverables in the tender specifications. Hello: the reason robotic did not even go into detail in the negotiations was that the owner requested the manufacturer to provide a loan for the carpark investment that then will be recouped via parking revenues at later years/

From user interviews and information given to us about handling capacity for events, it would appear that the Westfalia system at the Conrad, met the specification reference availability, although we were unable to obtain sight of actual records.

Applying the definition for availability used for the Conrad, to the data we were provided for the Robotic Parking system (above), gave availability for the system of 99.95%. The supplier told us that the main reason for malfunction was because the UPS protecting the control system against spikes in the electricity supply; was undersized and that if this was up-sized availability would be 99.99%. The owner however had elected to not pay for this upgrade, possibly because their opinion the achieved availability was acceptable; and certainly when we interviewed customers availability was never mentioned as a response to open-ended-questions.

By way of comparison:
On railways, reliability is often defined as kilometers travelled between major disruptions. In Singapore this was 133,000 train-km in 2015; which translates into about one major-disruption per year per train; Taipei achieved a reliability of about one major disruption per 3.6 years per train.

For automatic-parking a major disruption might reasonably be defined as one where a car, or more than one car, cannot be retrieved within 30-minutes of when the user requested it.

- When we evaluated the systems in Dubai; in-total the installed automatic-parking systems had operated nine-system-years.
- We heard of two breakdowns in the systems in Dubai where some cars were stuck for more than 30-minutes:
  - In one the fire-sprinkler system was set-off by a dust-storm when 64 cars were parked inside, this shorted the electrical system and took three days to resolve.
  - In another a servo-motor failed without-warning and it took a day to replace whilst twelve cars were stuck in the section served by the motor. If there had been automatic monitoring of the servo motors (present in two of the four systems), this should not have happened. Also in this section there was only one transporter, if redundancy had been provided by a second transporter only one car would have been stuck.

Assuming most major malfunctions were reported to us; that averages 4.5 years operation per major disruption per system. By that marker automatic-parking systems operating in Dubai, are more reliable than Taipei Metro.

9: Operations & Maintenance
Typically systems were operated by teams of typically seven technicians and attendants, working in shifts depending on operating hours. We were told that operating costs including cost of staff, utilities, and replacement equipment, were in the region of AED 3,000 ($800) per space per year. This compares quite favorably with the costs of operating below-grade multi-storage garages in Dubai which is typically AED 2,000 ($550) per space per year, mainly covering electricity costs for lighting and ventilation. Pallets systems are more expensive to operate and maintain than buggy systems since they are more complex. We were told by Sotefin that a total of 3,000 spaces in twenty locations in Milan, which are un-attended, are maintained by two technicians.

Operations and maintenance issues reported to us included:
- **Dust on light-activated sensors**
  Since it seldom rains, the air in Dubai contains fine dust and sand. The pallet systems are more complex than buggy-systems because the empty-pallets need to be handled; and these appeared to be more susceptible to dust; a good proportion of maintenance; was simply cleaning sensors; a better air-filtration system might resolve this problem;
- **Dust in card-readers**
  Some card-readers were sensitive to dust and failed, this can be resolved by selection of wireless technology, or devices that are proven in Dubai.
- **Un-expected failure of Servo Motors**
  Two of the systems in Dubai monitored the run-hours and condition of servo-motors in real-time so preventative maintenance was straightforward, and there were no breakdowns. In the other two, sometimes there were breakdowns, as mentioned above. Monitoring each element of the system in real-time increases the capital-cost; but this improves availability.
- **Vibration affecting PLC’s on transporters**
  We were told that on two of the pallet-systems the PLC’s on transporters failed frequently because of vibration, this was resolved by mounting them on rubber dampers.
Although no hard data on operations was provided to us, outside of one-month computer logs on one system (above), our impression from observation and conversations with operator-technicians and users was:

- As well as serving offices, the system at the Conrad is also used to handle large conferences and events with up to 350-vehicles processed in one-and-a-half hours. Processing vehicles efficiently at large events is a selling point for large conference facilities, and regardless of how many valet parkers are deployed, in Dubai at many large functions the wait for valet-parking is often more than 30-minutes; because regardless that the patrons don’t have to fetch their cars from a multi-story garage, the valet-parker does, and there are limitations on capacity of multi-story parking, plus there are communication issues relating to where cars are parked. We were told that at the Conrad the wait is typically less than ten minutes for functions.

- That said the system appeared to be more prone to minor break-downs and required a higher level of maintenance than the other pallet systems, it is likely that this may be in part because the main method of monitoring the system was CCTV rather than via collecting and displaying information on the status of every motor and transfer vehicle via the computer control system. The specification for this system recommended twelve gates; this was reduced to eight mainly because of limitations of space. Since the car park was never full when we inspected it, it was not possible to understand whether twelve gates might be required at some point in the future, so as to avoid choking during rush-hour.

- The system at Emirates Financial Towers was undersized reference throughput capacity and this was a source of complaints because the system choked in the rush-hour. It is possible the main objective on this development was to deliver nominal parking spaces, so as to meet the building code regulations; since 80% of the cost of an automatic parking system is in the handling capacity. We were told by the supplier that the design did have a provision for increasing throughput capacity by increasing the number of lifts from six-to-eight; it is likely that if this was done the issues with choking at peak times might possibly be resolved, although we were not told of any plans to do this. attention: the MAIN reason is that there is no robotic software implemented! The owner has chosen to implement his own software instead of Robotic’s Cimplicity platform. [please add this to be clear]

- The system at Ibn Battuta Offices was provided with four five lifts with space to increase this to eight lifts; also two of the seven levels were not used and were not provided with transfer vehicles. Apparently the plan from the beginning was to add capacity when the offices started to fill up and the lower throughput capacity became an issue. This decision was made in 2008 when it looked likely that there would be an over-supply of offices and resulted in a saving of 35% of the initial CAPEX. From the data we were provided (above), it would appear that the car-park which was only 50%-full at peak; would have capacity to provide sufficient throughput when the car-park was 100% full, after doubling the number of lifts. We were not told of any plans to do this.

- The system at Buildings-by-Daman was sized as specified (ten-gates and ten lifts for 800-spaces), and appeared to have no issues with throughput capacity. Although no hard information was provided to us, our impression, from talking to operator-technicians, was that the frequency of minor breakdowns was slightly more than in pallet systems.

10: Feasibility

The reasons why automatic parking might be considered for projects; instead of conventional multi-story; can be more complicated than a simple cost-per-space comparison of an equal number of spaces

With automatic it’s possible to build two-to-three times the number of spaces in the same volume. The feasibility of that option needs to be evaluated on a case-by-case basis and needs to account for cost, value, and the time-value of money, and these must be estimated quite accurately. Often this requires schematic design of both options. The complications are often the reason automatic is not considered, particularly because the analysis needs to be done at concept design stage, and at that point in projects, time costs money.
We were involved in the decision making on both the Conrad and the Buildings by Daman; so we saw how feasibility was evaluated. In both cases, the development team concluded the net-cost of going automatic, all things considered; compared to the net-cost of going conventional; was in one case almost half, and in the other case, almost one-quarter the net-cost of going with conventional multi-story or basement parking.

- **The Conrad**
  Although it was not the first one to get built, the first project that seriously considered automatic parking in Dubai was the Conrad. The circumstances behind the decision are rather unique and unlikely to be replicated elsewhere. It was all to do with a perception in the mind of the developer, about the value of land; which in retrospect may, or may not have been correct.

The reasoning was as follows:

- The plot of land for the Conrad was given to the owner by H.H. Sheikh Rashid Al Maktoum; thus there was no limit on how high the building could be or how much gross-floor area (GFA) could be developed; so long as the parking conformed to minimum Dubai Municipality standards.
- A decision was made early-on to build a hotel and an office. One reason for choosing this mix was because it was reasoned that the overhead of the parking could be shared; between office workers in the day, and hotel guests and particularly guests at conferences and functions, in the evening.
- It was also thought that good parking was important for a high quality building in Dubai, because at the time there was not great public transport, and in the summer there are limits to how far it is practicable to walk without becoming disheveled. In Dubai, a shaded car-parking close to the entrance to a building, in a reserved spot; can be rented for ten-times what un-shaded first-come-first served spot rent for, two-hundred meters from the entrance. Parking is real estate, and real estate is all about location.
- It was decided that the parking ratio should be 1:40 (one space per 40-m2 of GFA), which was the ratio stipulated by DIFC, but was more generous than the ratio of 1:60 stipulated by Dubai Municipality, this was effectively 1:35 because parking allocated for hotel guests could also be used by office visitors in the day.
- But unless basement parking was built, at an exorbitant cost, because of a high water-table; only 600 good quality parking spaces could be built, which meant that only 240,000 sq/ft of offices could be built, if the parking ratio was to be limited to 1:40.
- Even though the 600 spaces were above grade; they were expensive. It had been decided to put a luxury pool deck on top of the parking, so as to create “City-Oasis”, because in Dubai a significant portion of the hotel business is “business and leisure”, and beach hotels in Dubai get twice the REVPAR than city hotels. It was reasoned that by having an outstanding pool-deck, some of this business could be targeted; also in the cooler months alfresco dining is very popular, so a generous outside area can add value to a hotel.
- The quantity surveyors (Davis Langdon) worked out that after providing space for transformers, a mechanical room, and strengthening the building to support the pool deck; as well as providing large stairwells for evacuation of the pool area; each parking space would cost AED 135,000 ($37,000), i.e. AED 81 Million for 600 spaces. This was much more than was commonly paid for “standard” (lower quality) above-grade multi-story parking in Dubai – commonly thought then to be about AED 70,000 ($20,000) per space.
- Using automatic parking, 1,354 spaces could be built in the same volume, but the cost was high because of the requirement to support the pool deck on top, and also because the plot dimensions were not friendly for this concept. It was estimated each space would cost AED 144,000 per space ($40,000) for a total of AED 195 Million.
But it was reasoned that to compare the concepts, an allowance should be made for the fact that in prime offices, many of the parking spaces are “reserved”, so in practice the car-park is never 100% full. With automatic parking everyone is guaranteed a space, right in front of the entrance to the building, so the effective number of spaces is more.

When researching the concept, we had visited an automatic public parking built by Westfalia in the center of Vienna. Most patrons had season tickets, which guaranteed them a parking spot at all times. But 20% more season tickets were issued, than there were parking spaces, yet the car-park was never full.

So we figured 1,354 automatic spaces would provide effectively 15% more spaces than the same number of conventional spaces (i.e. effectively 1,557 spaces); in which case the cost per space was AED 123,000 ($33,000) – i.e. a little less than “conventional”.

All the same, the cost of building prime offices worked out at about AED 200,000 ($55,000) per 40m²; so the parking was costing more than half what the end-product was costing. That equation is one reason why many developers in Dubai build the minimum amount of parking they can get away with.

But by building 1,557 spaces (effectively), it would be possible to build 620,000 sq/ft offices; because the parking was the only constraint on how much could be built.

At the time land in DIFC, half a mile away, was selling at AED 400 per sq/ft of permitted gross floor area (GFA), and although there were no benchmarks outside of free-zones, the project real-estate consultants put a value of the land on the Conrad site at AED 250 per sq/ft.

So, it was reasoned, the options were:

A. Spend AED 81 Million and build 240,000 sq/ft of offices with parking costing AED 135,000 ($37,000) per space
B. Spend AED 195 Million to build 620,000 sq/ft of offices; with the parking costing AED 123,000 ($33,000) per space (effectively)
   - And get 380,000 sq/ft of GFA, worth AED 95Million – for free
   - Net-cost AED 100 Million for 1,557 spaces costing AED 61,000 ($17,000) per space after the increase in “land” was accounted for.

Also; by having automatic parking, and a lot of it, it became feasible to host large functions, where there is a demand for up to 500 cars to be parked or retrieved in one-and-a-half hours, which is impracticable if the parking is a multi-story.

So the hotel component was upsized to have two banqueting halls, so as to be able to accommodate Arabic weddings (male/female), and conferences where there is a conference hall, and then a banquet space above.

It was reasoned that if 30% of the added value was gained by the hotel component, also an automatic space, with the convenience of customers parking essentially right in front of the building entrance, created a better “sense of arrival” than negotiating an ugly multi-story car park.

By that logic one automatic parking cost effectively AED 40,000 ($11,000) per space, i.e. 30% of the cost of conventional.

It was a convoluted logic, much debated by the development team.

Whether the reasoning was valid or not, can still be debated. But for good or for bad, the reason automatic parking was chosen for the building was because the developer convinced himself that an automatic parking space, in this case, would cost much less than what a conventional multi-story parking space would have cost, all things considered.

Subsequent to this initial top-line analysis there was value engineering and negotiations with suppliers; and in the end only 1,000 spaces were built to support, 400,000 sq/ft offices. Then there was the financial crisis,
and land values halved. Today the Conrad offices are not full so the benefit of the “free-land” has not been realized...yet. However the offices do command a rent that is 50% more than other offices in the locality, where developers paid per space (effectively), twice as much for parking.

In retrospect, if the extra 200,000 sq/ft or so of GFA had been residential, which requires less parking and the main demand is at night, and only 240,000 sq/ft of offices had been built (and the big banqueting facility had been scrapped), the same logic could have applied to multi-story, and the return on investment might well have been higher. In other words, ten years after the decision was made to build automatic rather than conventional, it is still not crystal clear whether or not that was the best decision.

**Buildings by DAMAN**

For this project the developer had bought the right to build 1.6 million square-ft of Gross Floor Area; he could chose, what to build...offices, residential, hotel, or retail.

The building was in the Dubai International Financial Center (DIFC), which was designated a “free-zone” so unlike in most parts of Dubai; foreigners could buy the real estate, which meant the price of real estate was higher than outside of a free-zone. At the time the planning was done there was a market for “off-plan” real estate, and demand of off-plan offices was high, so this represented the highest & best use; with minimal market risk because units could be sold up-front (assuming the buyers paid their installments).

So it was decided to “max” the office space, and build 400,000 square ft of office GFA. The main constraint was building enough parking so as to meet the DIFC rules that stipulated one-space per 40m² of GFA, i.e. 1,000 spaces. The remainder of the development also had a requirement for parking, and also there was a limit imposed by the DIFC design guidelines on how high the parking structures could be above grade. The way the architecture worked out, the 1,000 office spaces would mean that three basement levels would be required if conventional parking was used. Since the water-table in the area was high; this would have been expensive, because the deeper a basement is, the more expensive it is per unit of area.

Opinions varied on how much the third basement would cost; from AED 130,000 ($35,000) per space to AED 150,000 ($41,000) per space.
- That cost was an up-front cost, the money would needed before anything else could be built
- Also basements often end up costing more than you thought they would; and they often leak; and the deeper they are the more likely they are to leak.

By going with automatic it was possible to avoid the third basement and “hide” the parking spaces under the offices so they were all above grade. This was achieved by persuading the DIFC planners that this was a “mechanical-room” and it looked indistinguishable from an office from outside. This was approved because the main objection the planners had to multi-story parking above grade, was that these can be ugly, and they often make access to buildings from street level, clumsy.

The estimated cost of the automatic system plus the civil works was about AED 100,000 ($27,000) per space, i.e. 30% or so less than basement parking.

But also, 70% of that money did not have to be spent until six-months before the whole building was finished. We were using a cost of funds of 7% in our calculations, for a four-year build that saving was worth AED 20,000 ($5,500) per space. With conventional basement-parking 90% of the cost is up-front, at 7% that meant for a four-year build using the average of the cost estimates; the cost of the money would be AED 35,000 ($9,500).
In the event it took eight years to build the building because of the financial crisis which resulted in off-plan buyers who put a deposit down at the peak, not paying their installments; so in the event, after taking into account the time cost of money; the automatic net-cost was about AED 115,000 ($31,000) per space; and the basement parking option, would have had a net-cost AED 200,000 per space ($55,000); almost double.

As with the Conrad; perceptions of cost and value of parking, depend on your vantage-point.

We were not involved in the development of the two other automatic parking garages that were built in Dubai so we can only speculate about the decision making. We did however have a few informal conversations with the real-estate company ASTECO, who were consultants on the Conrad project and attended the debates about parking, so they were aware of this analysis. ASTECO later went on to specify the concept for the Ibn Battuta offices which they developed. We also had informal conversations with the architects on the Emirates Financial Towers after they asked us for information on the concept.

Both of these systems were under-designed with regard to throughput capacity, in our opinion.

- **Emirates Financial Towers (EFT).**
  This system, manufactured by Robotic Parking Inc, has eight nine entry/exit gates and six lifts, which rule of thumb gives a theoretical peak throughput of 180 vehicles per hour (VPH). There are 1,200 spaces (one lift per 200 spaces); so in theory a full car park can be emptied in 6.7 hours. It is no wonder that the system clogs in the rush hour even when the system is not full, and at those times it can take the system 20-minutes to retrieve a car, it must be noted however that the owner decided to implement a software made by his own. Robotic Parking’s software analysis for this project shows a 350 CPH value which would translate to a 3.43 hour empty time..

  By comparison, for the Conrad and at Buildings by Daman; one lift per 80 spaces were specified initially although in the event the ratio was down-sized by cost-cutting when the financial crisis hit, to one-per 100 spaces at The Buildings by Daman, and one per 125 spaces at the Conrad; all the same these systems have almost twice the unit handling capacity of EFT which has one lift per 200-spaces.

- **Ibn Battuta Offices**
  The automatic parking has 765 spaces and four five lifts, i.e. one lift per 190 153 -spaces; there are 250 spaces above-grade under the building which comprises 400,000 sq/ft offices; giving an overall parking ratio of 1:40. The computer logs we were provided by the manufacturer, also Robotic Parking Inc, showed that the garage was never more than half-full (the offices were less than half full), so effectively the space/lift ratio was one to 95 76. As remarked above the system did not choke during peak hours, which suggests that 1:100 to 1:125 is probably a good ratio for offices in Dubai. It’s likely however that whenever the offices fill up, throughput will become an issue.

We can only speculate why these garages were under-sized with regard to throughput (in our opinion), we thought of three possible reasons:

- **Lack of information**
When we interviewed manufacturers of automatic parking systems in 2005/6; and also during our study in 2015, we were intrigued that information and/or guidance on what would be the optimal design throughput were conspicuously absent. Outside of the data provided to us by Robotic Parking Inc, during ten years, no other manufacturer has either published in the public domain, or provided us with any useful data/records of how well their systems work in practice with regard to throughput-handling capacity. Could it be possible that neither the developers, nor their consultants, were aware that throughput capacity is one of the most important criteria when specifying an automatic parking system? To this date – and in all of our history (except as ABMC) no planner ever did provide us a realistic peak traffic number! [you may bring about the difficulty of architects to provide such as they are not used to in multi-storey]

- **Circumventing parking regulations**

  It is conceivable that the developers believed that the parking ratios mandated by the authorities were too generous, and this amount of parking would never be used; so building the un-used parking would be a waste of money.

  If that was the case, then automatic parking would have been a perfect solution, because the cost of a space, over and above those that could have sufficient throughput capacity to empty a full car-park in two-to-three hours, is simply the cost of the space without the machinery to service it; which is about AED 30,000 ($8,000) per space for a pallet system and AED 40,000 ($11,000) per space for a buggy system; compared to AED 100,000 ($27,000) for spaces under large building, above grade, and AED 140,000 ($38,000) per space for basements.

  So in theory the submission of plans to the authorities for the building permit, could show the parking conforming to the mandated parking ratio. Except only half the parking could ever be used, in practice. Rather like building a multi-story car-park, and electing to save money by not building ramps between the floors. On one system we inspected in Korea we were told this was explicitly the strategy; the entrance to the automatic system was below the basement parking, difficult to access, and hardly used; built mainly just to achieve the specified parking ratio, which was much too generous.

  Singapore Parking Guidelines appear to have spotted this potential ruse. They stipulate there should be one lift per no more than 50-spaces in residential developments served by automatic parking. This was likely based on consideration of the traditional low-throughput parking systems built in Asia; and this could explain why there are no high-throughput automatic parking systems in Singapore; because for residential, one lift per 100-to-150 spaces is normally more than adequate using the modern high-throughput systems.

  The regulations could perhaps be improved if the throughput capacity was properly defined (percentage of cars that can be handled IN or OUT in one hour), assuming say 45-seconds dwell-time (the average time for users to park their car in the terminal, or to drive it out of the terminal); which is the number that was adopted for the Conrad sizing calculation, based on observation in conventional parking, and information provided by Robotic Parking Inc and Sotefin.

- **Just-in Time**

  It would be quite possible for both systems to retro-fit extra lifts and increase the numbers of transfer vehicles on each floor, so as to increase throughput handling capacity. Perhaps this was the plan?

  If so in retrospect, particularly at Ibn Battuta Offices, that would have been a brilliant strategy.
The cost of the upgrade would be about AED 15 Million for the 765-space garage. When we inspected the garage it was working fine, because the offices were half-full and therefore the latent under-design of throughput capacity was not an issue. Perhaps by 2019 the offices will have started to fill up, in which case, the handling capacity could be increased.

In the meantime the developer would have had the use of the AED 15 Million. In the interval, the installed spaces would have paid for themselves. Using say a 7% discount rate; in 2009 the net present value of defraying that expenditure would have been AED 15,000 ($4,000), to AED 30,000 ($8,000) per space on the initial investment, which we estimate would have been about AED 90,000 ($24,500) per space for system capable of handling 700 cars in under three hours, so the cost per space at the time of building in 2009 would have been AED 60,000 ($16,000) to AED 75,000 ($20,000), compared to AED 100,000 ($27,000) to build spaces under the building.

In practice, a developer never knows how much parking he will need in five to ten years time. Of course he makes estimates, but what if the building does not fill up as quickly as hoped? Or what if, for example, a shopping center is a runaway success, and the amount of parking required exceeds his wildest dreams?

Providing for such contingencies with concrete multi-story; generally means the extra parking that might be needed in the future must be built at the beginning, because it is under the building, and then stand idle for perhaps five to ten years in the future when it might be required.

With automatic the development plan can be to build the minimum that will be required in say Year-3; but provide space or structures to be able to accommodate the maximum imaginable in Year-10. Then as the building fills up, and it becomes clear how much parking is required; order the machines and install them; saving up-front CAPEX expenditure, getting better use out of available funds; and mitigating the risk of having calculated the future demand incorrectly, either too high or too low.

11: The Future

The four automatic parking garages in Dubai were completed by 2013. These systems were predicated by decisions made ten years prior. Since then no new systems have been built, and none are being built, the reasons:

- Possibly this is because construction of large offices in Dubai has slowed to a trickle due to over-supply.
- Also the value of real estate is less, so the value of extra parking spaces is less
- The metro system works well so demand for parking is less
- As outlined above, evaluating feasibility of automatic parking, compared to multi-story or basement, can be complicated. And typically, developers and/or architects cannot—or are not used to–perform such
- Also there are no off-the-shelf standard specifications or design guidelines that can be copy/pasted. Would make not much sense anyways as each project is very specific

The likelihood of more automatic parking getting built; depends in the first instance on the perceptions of value of extra parking spaces. So what is an extra parking space worth in Dubai?

- In Business Bay a in a less than half-full office development (today) an extra (reserved) parking space sells for AED 3,300 ($900) per year, and there is no waiting-list – rule of thumb value AED 33,000 ($9,000).
  - The building is about 30% occupied
  - Likely the parking will not be fully utilized for five-to-ten years
  - So the value of an extra parking space on this development turned out to be about minus AED 50,000 ($13,000) since money was spent ten-to-fifteen years before any utility was obtained from this extra space.
In retrospect a “wait-and-see” approach afforded by choosing automatic might have led to a better return on investment.

By contrast a prominent Dubai shopping-centre developer recently retro-fitted extra parking in a ten-year-old mall at a cost of AED 150,000 ($40,000) per space.

- The extra parking was presumably worth more than that to him. We were told of an internal analysis where the value of one extra space was pegged at AED 250,000 ($65,000), so the investment of AED 150,000 ($40,000) per space, for extra spaces; likely made perfect sense.
- There are many shopping centers in Dubai where parking is over-stretched and where extra parking could perhaps be provided at a net-cost less than AED 150,000 ($40,000) per space; particularly if this could be built in front of entrances, and thus provide a type of semi-valet parking; and the extra value of this was factored into the feasibility.

- Emirates Towers, an iconic office building at the center of Dubai’s central business district, is short of parking. The building is 100% occupied. To rent an extra space over-and-above what is allocated as part of the lease of an office costs AED 20,000 ($5,500) per year and there is a waiting-list.
- Rule of thumb that works out at a value of about AED 200,000 ($55,000) per space.
- But it’s often difficult to build extra parking next to or integrated with existing buildings
- Although clearly it can be a good idea to provide for that eventuality during design

- At Dubai Terminal One there is a choice, you can park directly opposite the departure lounge for AED 50 per hour ($13); you can park across the road, with a walk via an air-conditioned bridge taking five minutes, for AED 10 ($3), per hour; or you can park for free, one kilometer away.
- This illustrates that for parking, like all real estate, location is key.
- The expensive parking is always almost full
- If automatic had been used, or retro-fitted; that might also be always almost full

- Outside most metro stations in Dubai there are bicycle-racks, these are generally always full, illustrating that the utilization of this type of public transport can be increased by providing easy access over an area further than people are prepared to walk. But what if there was car parking at the metro stations?
  - Many studies have been done to estimate the value of persuading drivers to use public transport rather than clog up the roads; they are theoretical and include estimates of the cost of pollution, and the cost of having half the population sit in a traffic-jam for half-an-hour every day.
  - For our study of a planned metro system in Saudi Arabia, for good or for bad, we used a number of AED 100 ($27) per car per day, i.e. about AED 20,000 ($5,400) per year, which rule of thumb is worth AED 200,000 ($54,000); plus also parking can be charged, and users buy metro tickets.
  - Dubai RTA recognized that there is value to be had by providing “Park & Ride” and they built two 3,000 space parking structures; one in Rashidiya is well utilized, mainly by commuters from Sharjah, the other, near the Ibn Battuta Offices, is not, probably because the main commute at this location is the other way.
  - But what-if parking was provided at every metro station?
  - Particularly at metro station near to places people live?
  - The problem of course is that no land is available, or if it were to be made available, it would be expensive
  - Potentially automatic parking could provide a solution for one of the dilemmas of public transport in the Middle East, which is that the only people prepared to walk some distance in the hot sun, are people who can’t afford to buy a car; so one of the primary goals of public transport, getting cars off the roads at peak times, is negated.
Because, with automatic parking, on a small site, it is sometimes possible to build four-to-five times the spaces that could be built using conventional; these could be under the metro, on land that is currently not used, and that belongs to the metro.

A new development in automatic parking is driverless cars. For all cars manufactured after 2016 it will be possible for drivers to drop their car at the entrance to a building; the car would then drive itself to the automated parking Exit-Entry-Gate and park itself; and do the reverse for retrieval. Outside of the convenience; this will:

- Reduce the “dwell-time” for parking and un-parking a car, during which time the gate is occupied; from what is commonly considered the norm of 45-seconds to as little as 15-seconds. Assuming the “machine-time” is increased, this will mean that throughput per gate could increase by about 30% meaning that the numbers of gates required to service peak traffic flow would drop by 30%, making it easier to fit the parking into tight spaces (this would not markedly affect cost since the internal machine capability would need to increase).
- Also the gates could be much narrower since there would be no need for drivers to get out of cars inside the gate
- And quite possibly the driverless car could safely reverse into the gate (or reverse out), which would mean a cost reduction since there would be no need to turn the car round.

12: Lessons Learned

High throughput automatic parking makes most sense when:

- The value of a parking space is high
- And the more extra parking that can be provided, the better
- And the automatic parking can be built above grade
- Whilst the alternative using conventional parking is to put this under a large building or in a basement.
- And there is uncertainty about how much parking will actually be required, when the building fills up.

However there can be a much higher up-front cost during the design and planning stages when automatic is considered and a comparison is made with the option to build conventional multi-story. Instructing the architect to build to the mandated code, and as cheap as possible; takes less than five minutes of senior management time; but to do a thorough analysis of the option of using automatic, takes much longer. For example, at the Conrad:

- The technology was researched, existing installations were visited.
- An automation consultant was hired to write specifications.
- Demand and value were researched directly in the field - rather than relying on established industry guidelines or standards.
- The architect prepared schematic designs of option which were priced by a quantity-surveyor.
- Bid documents for design-build-operate, which is not a common format in the region, were written; and offers were solicited from suppliers.
- Cost/value analysis was done.
- And the management team debated the options, at length.

That whole process cost easily 1% of the estimated cost of building a conventional multi-story car park, i.e. about the cost of providing five-to-ten extra parking spaces; and the whole process took more than a year, although since the building was complex, and many other decisions were debated at length, for example the value/cost of 50m2 bays rather than 40m2 bays which was the industry standard in Dubai at the time.
Rule of thumb, in our opinion; it’s only if the net-cost of automatic; after taking into account the value of extra parking, i.e. of a higher quality than can be provided by multi-story or basement parking, works out on first inspection at less than 75% of the cost of the alternative, does it make sense to go through the process of making a detailed evaluation.

But if the difference is bigger, it’s arguably not necessary to go through the laborious analysis that was done for the Conrad and a back-of-an-envelope analysis can likely suffice. Of course with the systems built in Dubai no-one had ever done automatic like that, or for those reasons, before.

In Dubai, which has more high-throughput automatic parking spaces, than any other city in the world; surprising lessons have been learned.

References/Notes:
2. ABMC is a Dubai professional-license company, registered in 1999. The firm provides consultancy and interim management to support development of new concepts
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**KEYWORDS:**
Near-Field Communication; indoor parking; mobile applications.

**ABSTRACT:**
Interested in mobile applications continues to grow in both academia and the industry. We propose to integrate mobile phones into the existing parking guidance information (PGI) system in underground or closed areas. The current system relies on the driver to spot a green light-emitting diode (LED) indicating an available parking spot. Parking availability is decided from the output of an ultrasonic range sensor. We feed the signals from the sensors to an embedded and networked microcontroller accessed from the mobile phone using the hyper-text transfer protocol. The mobile phone draws a map to the next available parking as it receives real-time information from the sensors. To draw the map we need to identify the current location of the car. GPS is typically used in outdoor environment, but cannot be used to indoor environments. We place Near Field Communication tags around the parking area that are used to identify the current location of the car. The mobile system represents a novel application area of the NFC technology and its functionality is proven using a real world model. The proposed PGI system integrates with the fire alarm system and can be used to draw a map to the nearest exit.
An Optimization Model for Minimizing the Cost of Constructing Highway Vertical Alignments

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1 INTRODUCTION

There is an ongoing increase in private vehicle ownership around the world. In USA, private vehicle ownership has almost tripled from 0.286 vehicles per capita in 1950 to 0.799 vehicles per capita in 2011 (National Transportation Research Centre 2014). In New Zealand, private vehicle ownership has also sharply increased from 0.137 vehicles per capita in 1950 to 0.522 vehicles per capita in 2012 (Statistics New Zealand 1950; 2012). Similar sharp increase is also found in Australia where private vehicle ownership increased from 0.147 vehicles per capita in 1955 to 0.552 vehicles per capita in 2013 (Australian Bureau of Statistics 1955; 2013). In the United Arab Emirates, private vehicle ownership is 0.313 vehicles per capita and it is expected to increase further due to the high gross national income and unfavourable weather conditions during summer months (The World Bank 2011). The increase in private vehicle ownership has led to a matching increase in the demand for parking around the world. At the same time, the increase in land prices in most major cities in the world has resulted in decrease in the supply of conventional outdoor parking. Based on that, multi-storey indoor parking facilities are becoming more common due to their advantage of providing the maximum parking capacity per land unit area. It was also found that indoor parking facilities are more preferred by users due to their increased safety, security, and protection from adverse weather conditions (Van Der Goot 1982). Given the high cost of indoor parking facilities, there is need to maximize the utilization of the available parking capacity provided by those facilities. Smart parking management technologies provide a cost-effective tool to increase the efficiency of parking supply by maximizing the utilization of existing parking capacity. Parking guidance Information (PGI) technologies are an example of smart parking information technologies that guide drivers, who are seeking parking spaces, to the exact locations of available parking spaces in large parking facilities. PGI technologies increase driver’s satisfaction. They also have several other advantages in terms of reducing delay, fuel consumption, pollution, and traffic congestion associated with unnecessary traffic volume generated by drivers who are searching for available parking spaces. Furthermore, PGI technologies also have the potential to improve traffic safety within parking facilities and increase profits for parking operators (in case of indoor paid parking) by increasing revenues and reducing labour costs (Bayless and Neelakantan 2012).

Currently, the most common PGI systems consist of sensors mounted to the ceiling of the parking facility at every parking space. Each sensor detects if a vehicle is parked at the parking spot where the sensor is installed. A light-emitting diode (LED) connected to each sensor alters between red and green to alert the users about the availability of each individual parking spot. The drawbacks of this method are two-fold. First, the users are required to inspect the status of each sensor visually to find available parking spaces. This task may distract the attention of the driving users and affect their safety. Another major drawback is that users will always need an unobstructed line-of-sight to different sensors in order to find an available parking space.

In this paper, a PGI system is proposed to detect the parking spaces available in indoor parking facilities (either paid or free parking facilities), and based on the data collected the system can guide individual users to the closest parking spaces available with providing detailed routes to those available parking spaces through an Android-based mobile phone application. In order for the system to determine the closest parking spaces available to the user and to provide the detailed route to that parking space, the location of the user must be detected by the system. Given that the system is typically deployed in an indoor environment, the location of the user is detected by using near-field communication (NFC) technology where the user is asked to swipe his/her NFC-enabled mobile phone against an NFC receiver tag connected to the system. Different NFC receiver tags may be installed near different entrances to the parking facility and/or at other selected locations. In addition to this introduction, the paper consists of four sections where the first section provides literature review on the technologies utilized by the proposed system. The second section provides description of the system’s architecture with providing details on different hardware and software components. The third section provides details on system validation under different operating conditions. In the last
section, conclusions are provided along with a discussion on the system’s advantages and limitations and possible extensions for future research.

2 LITERATURE REVIEW

Several PGI systems have been proposed by utilizing different technologies. Cheung et al. (2005) proposed a monitoring system for parking facilities (as well as for roadway traffic) using wireless magnetic sensors placed on the pavement to detect the presence of vehicles. Benson et al. (2006) proposed an automated management system for parking facilities based on wireless sensing nodes together with a custom medium access control (MAC) and routing protocols with a Java-based server that interacts with a serial-forwarding program to acquire and process the data. Tang et al. (2006) proposed a similar intelligent car park management system based on motes-processor-radio (MPR) boards that are compatible with IEEE802.15.4 protocol. Moon and Ha (2013) proposed a similar car parking monitoring system based on a network of 3-axis Anisotropic Magneto Resistance (AMR) sensors. Yan-Zhong et al. (2006) proposed another parking management system based on a network of ultrasonic wireless sensors where each sensor is equipped with 8-bit microcontroller unit (MCU) processor with 7.3728 MHz system clock and 128 kilobytes in-system programmable flash memory. Each sensor is also equipped with low-power wireless radio frequency (RF) module that provides 19.2 kilobytes per second communication speed of data. Joseph et al. (2014) proposed a similar smart parking system based on a network of wireless light sensors placed below the ground surface of the parking facility. The sensors detect the presence of vehicles by measuring the amount of ambient light with relaying the data into a central supervisory system (CSS) in the form of 2-byte data packets using IEEE802.15.4 protocol.

Based on the above literature review as well as on other references (Bayless and Neelakantan 2012; Ozdenizci et al. 2010), a PGI system typically consists of a network of sensors installed at entrances and exits or at individual parking spaces. Available sensor technologies include inductive loops, machine vision, ultrasonic, infrared, microwave, laser, and light sensors. The sensors detect the presence of vehicles and collect data regarding the number of occupied and available parking spaces. The data can be relayed to a central computer for processing and messages are therefore relayed to the users regarding parking availability through different types of media that may include static or variable message signs, mobile phones, radio, the Internet, or in-vehicle navigation systems. Depending on the technology used, the messages relayed to the users may range in the level of sophistication. They may be basic messages that inform the users whether the parking facility is “empty” or “full”. They also may be more-detailed messages that inform the users about the total number of available parking spaces. The most detailed messages may inform the users about the exact locations of available parking spaces.

NFC technology has the advantage of increasing communication capabilities of mobile devices, primarily smart phones, without draining their batteries. Given its short range of communications, which is usually in the range of a few centimetres, NFC technology is more secure than comparable technologies such as Bluetooth. With the increasing number of NFC enabled-devices, the number of NFC applications has also drastically increased in the recent years (Ozdenizci 2010). Unlike Bluetooth, NFC operates in much shorter range and requires significantly less power. Even though the bandwidth capability of Bluetooth is larger, the setup time for NFC is significantly less. This makes NFC suitable for a wide range of applications. NFC was designed initially to enable secure payment with mobile devices. Consequently, one of the most common usages of NFC is in the domain of payment (Pasquet et al. 2008). Generally, the wireless proximity technologies such as NFC and Bluetooth lend themselves naturally for contextual content dissemination. Kaye et al (2012) used NFC to track a water purification initiative in Haiti. They provided community workers with NFC-enabled mobile phones to increase the reliability and accuracy of data collection in the field when visiting households to check on their adherence to water purification. NFC technology has also been used to support attendance supervision systems in schools to replace manual roll call in order to save time (Ervasti et al. 2009). NFC technology is also implemented at several museums to provide visitors with different facts related to the displays (Blöckner 2009).

3 SYSTEM ARCHITECTURE

The system consists of hardware and software components that are fully integrated together to provide real-time parking information to potential users with also providing detailed routes to every user to the closest parking space available to that user. As illustrated in Figure 1, the system consists of a network of parking sensors that utilize ultrasonic technology to detect the presence of parked vehicles. The sensors are connected together to a microcontroller board that collects and processes the data to select the closest available parking space for every individual user. The microcontroller relays the processed data to the user through an Android-based mobile phone application. The system
detects the location of the user in the indoor environment by using near-field communication (NFC) technology where the user is asked to swipe his/her NFC-enabled mobile phone against an NFC receiver tag connected to the system. As soon as a parking spot becomes available, the map on the mobile application is updated to show the available parking spot and guide the user to that spot through an optimized route.

The sensor used is mounted to the ceiling of the parking facility. It detects vehicles by sending and receiving an ultrasonic beam to measure the unobstructed distance below the sensor. The unobstructed distance is reduced when a vehicle occupies the parking spot below the sensor. The status of sensors is relayed to a microcontroller through a data bus with tri-state buffers as shown in Figure 2. The data bus is controlled by the microcontroller and used in a round robin fashion to pull the status of all the parking spaces in groups. Using this technique, the system can be extended to accommodate any number of sensors. The microcontroller Ethernet port acquires an IP address and listens for connections by mobile devices. Requests by users are made by swiping the user’s NFC-enabled mobile phone against an NFC receiver tag connected to the system, and the system responds to the request by sending real-time information to update a pre-loaded map on the user’s mobile phone.

Figure 1. Overview of the proposed PGI system.
4 SYSTEM VALIDATION

To validate the proposed system, the research team designed a prototype model of a two-story indoor parking facility. The model was built to a 1:40 scale as per the conceptual design shown in Figure 3. The actual built model is shown in Figure 4. Every parking spot was equipped with an LED and an ultrasonic-sensor. All sensors were connected to a microcontroller board that controls the system. The printed circuit board of the microcontroller is shown in Figure 5. The complete assembled system is shown in Figure 6. The software component collects sensor readings regarding the availability of all different parking spaces and relays those readings to end users through an Android-based mobile phone application. A token-ring algorithm was employed to traverse all the sensors in structured groups with identifying the status of each sensor in each group accordingly. This structure has the potential to make the system scaled more easily with integrating any existing parking assistance systems.
Different screen shots of the developed Android-based mobile phone application are shown in Figure 7. In Figure 7(a), the application is launched by clicking on its icon. The application can also launch automatically once an NFC tag is scanned. In Figure 7(b), a real-time updated map of the parking facility shows that all parking spaces are available. In Figure 7(c), a parking space is identified by the system. The space is located behind the user and therefore it is outside the user’s visual field. The space is shown to the user on the map. The application also shows a detailed route to the identified parking space. The route shown to the user was created in accordance with the driving rules applied inside the parking facility (e.g. one-way vs. two-way driveways). Those rules are pre-loaded to the application. In Figure 7(d), another parking spot is identified by the system once became available. Since the new available parking space is closer to the user than the one identified in Figure 7(c), it is suggested to the user by the application. It is noteworthy to mention that the screen shot shown in Figure 7(d) was obtained by removing an actual model car from the corresponding parking space in the prototype model. This is to validate the developed model and ensure that it automatically updates the map and selects the nearest parking space available. The developed model has been validated again by making a closer parking space available than the one shown in Figure 7(d). Consequently, the model updated the map and selected the closer parking space, which is shown in Figure 7(e). In Figure 7(f), two parking spaces are
available where the first one is located on the same side of the user behind his/her location, and the second one is located on the other side of the parking facility. Given that all driveways at the parking facility shown are one-way, the system selected the second parking space since the driving distance to it is shorter than that to the first parking space regardless of the walking distance.

Figure 7. Screenshots of the developed Android-based mobile application.

5 CONCLUSIONS

In this paper, a smart parking system is proposed to provide real-time parking guidance information for users of indoor parking facilities. The proposed system includes a series of parking sensors that detect the presence of parked vehicles. The sensors are all connected to a microcontroller board that collects the number and locations of available parking spaces, selects the closest available parking space for every individual user, and relays the information to the user through an Android-based mobile application. The system detects the location of the user in the indoor
environment by using near-field communication (NFC) technology where the user is asked to swipe his/her mobile device against an NFC tag connected to the system. The tags can be located at parking entrances as well as at other desired locations. The system has an algorithm to select the shortest driving route to the available parking space according to the driving regulations applied within the parking facility. The system has been validated by building a scaled prototype of the system that includes all hardware and software components. The software component (the Android-based mobile application) is fully functional and has been tested and validated under different scenarios. The developed mobile application does not include payment module since most indoor parking facilities in the United Arab Emirates are free of charge. However, the system can also be extended to calculate the parking time and the corresponding parking fee. This may be achieved by registering the time the user enters the parking facility and swipes his/her mobile phone against the NFC tag. Registering the time the user exits the parking facility may be achieved by asking the user to swipe his/her phone to open an exit gate after paying the parking fee calculated by the system.

The proposed system has the potential to increase customer satisfaction and safety as well as reducing pollution and fuel consumption. The system also has the potential to increase overall utilization of the number of parking spaces with increasing revenues for parking operators in case of paid parking. If supplemented with automatic entry/exit gates that can be controlled by the system, the system can be fully automated with minimal manpower needed, which significantly reduces the costs of operating the parking facilities. The hardware components used by the system are all low-cost components when compared to other smart parking systems and they provide flexibility and the ability to be customized to any size of parking facilities. The system can detect available parking spaces and guide users to those spaces, even if those spaces were not within the visual field of the users (for example in the opposite side of the parking facility or even on a different level of the parking facility). There are also added benefits to the proposed system. Not only does it greatly assist in finding a parking spot, it also directs drivers to the exit in case of an emergency, which is an important safety feature given the nature of the underground parking.

ACKNOWLEDGEMENT

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- New Zealand Year Book
- Australian Bureau of Statistics


**PAPER TITLE**

INNOVATIVE APPROACH TO DEVELOP SMART PARKING SOLUTIONS ON PPP FORMAT

**TRACK**

3.4 Innovative Funding Mechanisms (incl. PPPs & BOTs)

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<td>ITNL Infrastructure Developer LLC, Dubai</td>
<td>UAE</td>
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**KEYWORDS:**

Public Private Partnership, Automated Car Parking, Private Financing, PPP, Infrastructure

**ABSTRACT:**

With oil prices plummeting, budgets shrinking and getting reallocated across sectors, Governments within the Middle Eastern Region are looking for alternative approaches to meet the shortfall in infrastructure spending. Public Private Partnerships (PPP’s) have been implemented in the Middle East since the early nineties on ad hoc basis, without the support of a PPP law or government policy in place. The announcement of Dubai’s PPP Law No. 22 of 2015, an initiative aimed at developing infrastructure by fostering partnerships between government and private sector will help circumvent large initial capital investments and amortize the cost of the project over a longer period of time. Taking this initiative further, consortium led by (ITNL Infrastructure Developer LLC) IDL conceptualized the ‘Dubai Supreme Court Project’ first of its kind PPP project to provide 200,000 sqft of office space and parking facility in excess of 1200 cars within a portion of existing Courts car park area which currently has 250 car parking spaces. This could be possible by implementing an innovative financing model and state of the art car parking technology.

The Project being a unique concept received the blessing of H.H. Sheikh Mohammed Bin Rashid Al Maktoum, Ruler of Dubai and Prime Minister of UAE. The first PPP Project in Dubai Transport Infrastructure sector will be implemented on DBFOT basis would create value at no cost to the Government and will mark the beginning of a new era in developing infrastructure on Private Financing and has undoubtedly got a replication potential across the region.
1. Introduction

Infrastructure has always been the fundamental aspect of the business environment, it promotes trade and commerce and generates employment. Over the past decade, Middle Eastern region oil exporting countries enjoyed large fiscal surpluses and rapid economic expansion on the back of booming oil prices and working aggressively to build state of the art infrastructure across the cities. However, with oil prices plunging in recent years, surpluses have turned into deficits and growth has slowed down, this has started raising concerns about Government’s ability to fund infrastructure projects to meet the demand supply gap. Low oil prices and deepening fiscal challenges continue to weigh on economic activity in the region and accordingly, these countries are promoting private sector participation in the implementation of infrastructure projects.

As per estimates, the Middle East and North Africa (MENA) region will need between USD 75 Billion and USD 100 Billion of investment per year over the next 20 years to meet the growing demand of world class infrastructure and to keep pace with the fast growing economies. Government’s inability to fund this huge shortfall has led to adopting a proven method of developing and financing the Public sector services through the Private sector participation under the Public Private Partnership (PPP or P3) format.

Recourse to PPP can help meet the budgetary constraints while simultaneously improving experience and capacity in not only traditionally critical sectors such as transport and renewable energy but also upcoming sectors such as real estate, parking, smart cities etc. The ‘Dubai Supreme Courts Project’ is the first PPP project to be developed in Dubai under the newly enacted PPP Law of Dubai and approved under the vision to transform Dubai into the smartest city in the world. This paper outline the challenges and opportunities in the region to develop the infrastructure projects on PPP format and specifically refers to the experience while developing the Dubai Supreme Courts Project.

2. PPP as Preferred Project Delivery Approach

a) Concept of PPP

PPP’s broadly refers to the long-term contractual partnership between the Private sector and Public sector entity with the aim of providing services which were traditionally delivered by the Public sector. While PPP’s were originally thought to be a derivative of privatization, there is a growing consensus that PPP’s are a type of collaboration to pursue common goals while leveraging joint resources and capitalizing on the respective strengths of the Public and private sector (Sambrani, V. 2014).

<table>
<thead>
<tr>
<th>Sources</th>
<th>Definitions</th>
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<tr>
<td>Her Majesty Treasury, UK</td>
<td>An arrangement between two or more entities that enables them to work cooperatively towards shared or compatible objectives and in which there is some degree of shared authority and responsibility, joint investment of resources, shared risk taking, and mutual benefit.</td>
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<td>The World Bank</td>
<td>The term “public-private partnerships” has taken on a very broad meaning. The key elements, however, are the existence of a “partnership” style approach to the provision of infrastructure as opposed to an arm’s-length “supplier” relationship…Either each party takes responsibilities for an element of the total enterprise and they work together, or both parties take joint responsibility for each element…A PPP involves a sharing of risk, responsibility, and reward, and it is undertaken in those circumstances when there is a value-for-money benefit to the taxpayers.</td>
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Sources | Definitions
| --- | ---
| European Commission | A partnership is an arrangement between two or more parties who have agreed to work cooperatively toward shared and/or compatible objectives and in which there is shared authority and responsibility; joint investment of resources; shared liability or risk-taking; and ideally mutual benefits.

| Canadian Council for Public Private Partnerships | PPP is a cooperative venture between the public and private sectors, built on the expertise of each partner that best meets clearly defined public needs through the appropriate allocation of resources, risks, and rewards.


For the purpose of this paper, PPP would mean a contractual arrangement between the Private sector and Public sector to achieve a common objective of developing infrastructure projects on Design, Build, Finance, Operate and Transfer basis.

b) Regions Experience with PPP Projects and Associated Regulation

Until now, a selective set of PPP projects in the Middle East region has been implemented in the traditional sectors primarily being energy and few more across water and transportation. A summary of the PPP experience amongst the key countries of the region is listed below:

Oman:
The first project to follow the PPP model in the Gulf region was the Oman’s Al Manah Power Project plant which was implemented in 1994. However, Oman did not actively leverage on its first PPP experience, well across the other sectors. Oman has now recognized the importance of PPP in developing projects and promoting economic growth and is currently drafting its PPP law under its recent five-year plan and will have a central PPP unit to manage the procurement process. Omani Government is also identifying a possible social Infrastructure project to test the proposed PPP framework. Other projects identified for PPPs are Oman Rail, Port Sultan Qaboos, Port Khasab, South Batinah Logistics Area, some fisheries projects, Ad Dhahirah Economic Area and Shinas Port.

Bahrain:
Bahrain does not have a PPP law but is believed to be working on it. There are a number of projects in the offing like Bahrain Saudi Causeway Project, the affordable housing, healthcare, integrated waste management etc. However, without a proper legislation in place, full-fledged implementation of projects on PPP basis will definitely be a challenging task.

Qatar:
Qatar is working on its draft PPP law which is due to be released soon. The new PPP law aims to expand the use of PPPs’ into both economic and social infrastructure sector. Qatar signed its first PPP project with a Japanese-led consortium to develop the $3bn water and power project at Ras Laffan (Warren, L. & Whyatt A. 2016). Qatar has indicated a wide variety of projects ranging from transport, sport, health education, hospitals and housing that it may be interested to develop on PPP basis.

Saudi Arabia:
Saudi Arabia has also expressed its intention to welcome foreign investment as part of its ‘Vision 2030’ statement, which is aimed to end Saudi Arabia’s dependency on oil by 2020 and to diversify its economy. While adopting a legislative PPP framework to be consistent with these laid objectives, no plans to adopt one have yet been announced. It remains to be seen whether Saudi Arabia will more formally embrace PPPs by implementing some form of legislative framework. So far, select airport PPP project has been successfully completed in the Kingdom. The projects likely to be developed under PPP model include schools, social housing, airports, rail and road, solar and wind.
Kuwait:
Amongst the GCC nations, Kuwait has been the most forward country in establishing a PPP law in 2008 and the declaration of the new PPP law 116 of 2014 and its executive regulations strengthened the PPP programme’s institutional framework and offered new benefits to these projects, including tax and fee exemptions. It also established the Kuwait Authority for Partnership Projects (KAPP) – previously known as the Partnerships Technical Bureau – as the main body responsible for implementing the state’s PPP programme launched in 2008. Unfortunately, due to poor execution methods, the PPP schemes have not really taken off the way it should have. Nevertheless, Kuwait has signalled its open-mindedness to continue working on its PPP framework in order to align them to international standards to attract investors.

UAE:
Over the past decade, despite the severe financial crisis, UAE in general and Dubai in specific has built a reputation of being one of the most stable & business-friendly destinations in the Middle East. Keeping up with its image of providing the best of class services, Dubai has plans to deliver a state of the art infrastructure across existing and new areas of development. However, in wake of the ongoing economic scenario, this seems to be a challenging task to meet its ambitious programme of investment and considering this Dubai has started looking at various options and has expressed its interest in PPP mode of procurement. In order to improve private investments across infrastructure development, Dubai has recently enacted PPP law no.22 of 2015 aimed at achieving socio-economic development in the Emirate. This move will help the government focus on the implementation of strategic projects in a more effective and efficient manner and replication to similar initiatives in the other Emirates of UAE.

Table 2: Middle East PPP Market Summary

<table>
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<tr>
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<th>PPP Law</th>
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Source: K&S and MENA Private Equity Association: PPP Forum (Burbury, T. 2016)

3. Parking Projects

a) Parking in Middle Eastern Cities

Development in the Middle East, particularly UAE, has boomed over the past few decades. This rapid growth and boom have led to a high movement of people, goods and services in the country. Owing to the growth in economy and population there has been a surge in the density of vehicles. In most cities in the UAE, a typical urban setting is characterized by large, mixed-use city block or sectors. Most commercial and residential areas do not have sufficient parking and the shortage of parking space in many areas in the UAE has led to serious concern among customers. In order to alleviate the existing parking problem and to accommodate future demands a number of parking demand studies have been undertaken which has indicated a huge supply-demand shortfall in the availability of parking spaces. In UAE, authorities in Dubai and Abu Dhabi
have identified several areas lacking the required minimum parking spaces and those could be developed as multistoried parking projects.

Private Paid parking lots have mushroomed in many parts of cities in the Middle East Region. These parking lots cater the many demands of drivers who continuously scramble in search for parking lots. With such a huge shortfall, developing Parking Projects on private financing basis seems to be emerging opportunity in the region with Government willing to leverage on exploiting land in order to create potential revenue streams (Dla Piper). With a new PPP law in place, the Private players in Dubai have expressed their interest in developing non-traditional sector projects such as ‘Parking’ on PPP basis.

b) Smart Parking Systems

Faced with increased Environmental and Economic pressure on city transportation, along with severe space constraints, cities all over the world are looking at new technologies and innovative approaches to develop parking to fulfil their current deficit and to meet the future demands. Smart Parking can be considered as a label for technological innovations and efficient services monitoring and management of parking within an urban mobility strategy.

Smart Parking Systems has many definitions, however, the term used in this paper refers to Smart Parking as an Automated Parking System (APS) designed to minimize the area required for parking cars. The automated parking system provides parking for cars on multiple levels stacked vertically to maximize the number of parking spaces while minimizing land usage. The APS generally utilizes a mechanical system to transport cars to and from parking spaces and avoid the driver to drive the car within the parking area in search of parking space. This resulted in elimination of the circulation space used in a traditional multi-level car parking area. APS is also generically known by a variety of other names, including automated car parking system, mechanical parking, and robotic parking.

Smart parking systems have an array of advantages. The most important advantage of smart parking systems is its implications on the environment. According to a study carried out in the USA in 2007 drivers in Los Angeles drove for more than 950,000 miles, emitted 730 metric tons of Carbon Dioxide and burned 470,000 gallons of fuel in search of parking. However, these problems can be reduced with the help of smart parking. Studies indicate that 10-40% of congestion in the cities are caused by vehicles looks for a parking space. Further, congestion also leads to increased fuel costs, pollution, lost economic opportunity and is detrimental to the quality of living. For developers looking to develop smart solutions for Smart cities, Smart parking presents a prime opportunity to do so. (Woods, E.). Considering these advantages, the oil rich Middle East countries are very keen to adopt the smart parking systems in their cities thus reducing the car emission by a significant amount while parking operations. Use of APS opens up more than 50 % of the space otherwise used in conventional parking and utilizing the same for either investment or for community facility purposes.

c) Potential for developing Smart Parking Projects using PPP Approach

Parking provision is a commodity in the regional cities and for private parking areas, the pricing is based on demand supply gap. Considering increased land valuation due to the high density of development in the Middle Eastern cities, government agencies are proactively seeking private investors to build public parking along with ancillary facilities on a commercial format. To make the projects viable investors need to deploy technologies those are highly space efficient and has an ease to use for parking users. Automated Robotic Parking systems fit well in this market landscape.

Using this approach, government land currently being used for conventional parking areas that take roughly 30-35 sqm per car space can be converted into Automated Robotic Parking Garages that can take approx. 12 – 15 sqm of area per car space. The space that is saved could be used for commercial exploitation purposes creating an additional revenue stream for the investor or providing more parking spaces thus substantially increasing the parking revenues. Dubai being the champion in adopting modern technologies took a lead in this area and approved a robotic parking project for an important public office building using PPP approach. This Project is a pioneer in (i) Use of latest technology in car parking space and (ii) Use of PPP framework in implementing urban projects, thereby bringing value to all the stakeholders.
4. **Dubai Supreme Courts Project**

**a) Background**

Dubai Courts is the seat of judiciary for the Emirate of Dubai and have other associated services where regular visitor’s inflows is a daily phenomenon. With a rapid pace of economic activities in Dubai over the last decade, the Courts are getting busier and the existing Court is facing serious constraints in the expansion of office area, improvement for public services and facilities. Dubai Courts is also facing extreme parking shortage through courts operating hours and were keen on exploring efficient parking solutions along with convenient facilities for courts users and visitors.

In 2014, a consortium led by IL&FS Transportation Networks Limited, Dubai, presented a proposal to resolve parking problems by developing a robotic multilevel car parking facility in the existing courts premises on a private finance initiative basis. Considering the project viability with no direct recourse to the government, the project scope was evolved over intense consultative working sessions over the last one and a half year. Finally the project is conceptualized as the ‘Dubai Supreme Courts Project’ with an objective to convert a portion of the surface employee car park area with 250 car spaces, within the Court premises, into a complete and integrated multi-use development of ~200,000 sqft to house the esteemed Supreme Court, its ancillary services and a state-of-the-art Robotic Car Park having a capacity of more than 1200 car spaces on Public Private Partnership (PPP) framework.

The project is first of its kind PPP project which addresses two major issues faced by the Dubai Courts viz; shortage of office space and severe shortage of parking. The Project is conceptualized over a period of eighteen months of consultative discussion process with Dubai Courts and various other Dubai Government entities including Dubai Finance, Dubai Legal, Dubai Real Estate Authority and other relevant stakeholders. Finally, after enactment of Dubai PPP law in 2015, Dubai Courts awarded the Project in 2016 to the Consortium on PPP basis for a concession period of 30 years. The project will have a robotic parking system and the technology is sufficiently localized to adapt itself to local environments.

The Project will be implemented on Design, Build, Finance, Operate, Transfer basis at no cost to the Government and has been conceptualized considering local heritage and traditional Arabic design philosophy with due importance given to local environmental factors in the surrounding creekside area.

**b) Project Structure**

In a PPP model, there is no ‘one size fits all’ concept owing to the flexibility of the type of arrangement that can be made between the private party and the public sector entity. ‘Generally, trust, openness and fairness are basic foundational underpinnings of successful PPPs’ (Jamali 2004). Based on this model the transaction structure was evolved with mutual consent with the government and the project is to be delivered through a Special Purpose Vehicle (SPV) formed by the Consortium members.

Figure 1: Project Implementation Structure
c) **Key Concession Terms**

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<th>S. No.</th>
<th>Term</th>
<th>Description</th>
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<td>1.</td>
<td>Concessionaire</td>
<td>The Project to be implemented through an SPV as per applicable laws to undertake the Project on Design-Build-Finance-Operate and Transfer basis under a Public Private Partnership (PPP) basis. Dubai Courts shall provide all reasonable support and assistance to the Concessionaire in procuring applicable consents and permits required from any government body for the implementation and operation of the Project. The Consortium shall be responsible for the Establishment of SPV, Project Financing, day to day management of the SPV, reviewing the design and undertaking construction activities, Project Monitoring, Project Insurance, Operation and Maintenance of Project excluding Supreme Court Area.</td>
</tr>
<tr>
<td>2.</td>
<td>Concession Period</td>
<td>The Concession Period shall be 30 years including the Construction Period of 30 months. Concessionaire after completion of the Concession Period shall handover the entire development area with automated car parking system in line with the handing over requirements detailed in the Concession Agreement.</td>
</tr>
<tr>
<td>3.</td>
<td>Project Scope</td>
<td>Schedules to the agreement shall clearly define the scope of work to be undertaken including operations and maintenance scope and related performance standards.</td>
</tr>
<tr>
<td>4.</td>
<td>Operation and Maintenance</td>
<td>The Concessionaire shall be responsible for Operation and Maintenance of the entire Project including the car parking technology excluding the Supreme Court Area. The Concessionaire shall be responsible for ensuring compliance with pre-agreed performance parameters, failing which, a penalty provision (as detailed in the Concession) shall be levied on the Concessionaire.</td>
</tr>
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</table>
| 5.     | Concession Fees          | Under this Public Private Partnership Project, against a 30 year lease of 7,000 sqm plot, the Consortium would work as a partner to the Dubai Courts and will share a Profit of 25% for first 8 years of operations & 40% for the balance period. In addition the Concessionaire will also provide  
  • Minimum of 250 spaces to Dubai Courts at No Cost within the 1,232 automated car spaces.  
  • 47,576sq feet of Supreme Court Development Area (Supreme Court Area) at no cost to Dubai Courts. |

**d) Risk Sharing Mechanism**

Under the project implementation framework, the responsibilities for designing, building, financing, operating and maintaining are bundled together and transferred to the project SPV which have in turn transferred the risks to the equity partners. Due to the bundling of all sub component of a project into a single contract, the entire risks related to project delivery is passed on to the Concessionaire (SPV) and in this
Project, these risks are assumed by the project sponsors which is different from a typical risk sharing in any DBFOT contract. Following are the broad risks that are assumed by SPV in this project:

- Design Risk
- Construction Risk
- Programme Risk
- Revenue Risk
- Force Majeure Risk
- Financing Risk

As per the project structure, the project sponsors have committed to assume all the risks as listed above. The consortium members have taken full responsibility for delivery of the project within the cost and time defined in the Concession Agreement. Any cost overrun for reasons attributable to the consortium members shall be funded by the consortium members with no liability on Dubai Courts. Time overrun (for reasons attributable to the consortium members) shall invite levy of penalty by Dubai Courts on the Concessionaire.

The SPV shall enter into binding service level agreements with project sponsors specialised in their respective fields for ensuring the performance standards as defined in the Concession Agreement. The respective binding service level agreements would be executed with the respective project sponsors thus mitigating any time overrun, cost overrun and performance risks on part of the government.

e) Replication Potential

This Project being the first Design, Build, Finance Operate, Transfer Project in the area of smart parking solution to a public sector office has clearly got a replication potential across the region. Most of the Government offices which caters to Public services are facing parking problems. These offices are generally located within the heavily built-up areas and has limited options to expand the public facilities. The Parking requirement at these offices are much higher than the supply and further, the room for expansion is very limited due to space constraint. The solution to this problem is to use space efficient parking technologies. The Proposed Project includes the application of technology that uses 1/4th of the area of the conventional parking system. Thus resulting into providing of more than 1200 spaces where there are currently 250 spaces and also creating more than 200,000 sqft usable office space within the same plot.

f) Key Challenges in the transaction

a) Co-operation between the parties:

A major challenge of PPP is co-operation between the public and private entity to achieve a common goal. It is a known fact that private agencies are profit oriented whereas public institutions often work to meet the basic necessities for the people. Engaging two parties with conflicting interests to put their ideas and resources together is difficult. However, with a strong Agreement that binds two parties defining their roles and responsibilities and with proper legislation these challenges can be tackled proficiently. In the Dubai Courts Concession, both the parties in the initial level realized the importance of cooperation and gradually during the process there was a synergy between the efforts of both parties and the project business case was reviewed in a transparent manner. Dubai Courts being a profit share partner capped the cost with any overrun to be funded by project sponsors off the balance sheet of project SPV.

b) Project Financing:

For initial sets of PPP Project, it is expected that the Government to provide some comfort to the project lenders as they perceive a risk due to no track record of government in managing such projects. In the absence of any support from the government, lenders seek some extra comfort from the promoters and that results in the increased project cost or delayed project financing. In the Dubai Courts Project due to non-availability of any government backed security instrument, local lenders have seen this transaction as an unsecured project. Finally, the transaction was concluded with some experienced lenders on the basis of project recourse funding with guarantees given by the project promoters. From a risk perspective, the project is perceived as a Real estate project rather than as an Infrastructure project. This led to lenders classifying
such projects as high risk leading to higher financing costs. This could be mitigated if projects with the prime objective of resolving parking issues could be classified as infrastructure project with required support from the government by way of providing some comfort to the lenders and allowing securitization of developed assets.

c) Understanding of PPP amongst various stakeholder:

Since the very beginning, Governments across the GCC have funded large infrastructure projects from public sector budgetary allocation. Also, transferring the project rights to the concessionaire to develop and operate the project does not fit within the existing procurement and project approval rules. Dubai Supreme Courts Project being the pilot project has faced challenges and a time-consuming process with various authorities’ approvals, however during the process these authorities have become familiar with the project structure and it is likely that for the other projects in pipeline, approval process would be a seamless going forward. Developing a pilot project with a project champion is key to the success of implementing such projects.

d) Legal framework

Although there are strong indications from governments in the region to enact a robust legal framework system to facilitate PPP Projects, the legislation development in most of these countries are still in its very nascent stages and will take time for private investors to develop some level of confidence. Dubai has enacted the PPP Law, however in the absence of a central PPP unit it is very challenging to present a consolidate PPP pipeline to potential investors, and it is desirable to have a well thought-out framework for PPP to attract foreign investment. It is also expected that the legal framework should also address the concerns regarding the SPV shareholding structure due to foreign ownership restrictions, foreign investment protection, dividend repatriation and mechanism of addressing termination events.

e) PPP regulator:

Under the newly enacted Dubai PPP Law, each government department can create their own PPP unit rather than a central PPP unit. This approach has both advantages and disadvantages. Due to the nature of PPP projects, which primarily involves long term association between a public and private entity, there needs to be an independent PPP regulator with a sound regulatory policy that ensures a smooth and transparent implementation of PPP projects.

5. Conclusion

With oil prices showing no signs of recouping in the direction of previous highs and with the worldwide economic challenges, it would be wise to look at alternative methods of funding infrastructure projects in the Middle Eastern Region and more and more government should look into developing projects on PPP basis. With the announcement of Dubai’s PPP law no.22 of 2015, Dubai has cleared its intention of developing a firm framework for execution of future projects on PPP basis. However, they need to work in setting up a detailed PPP framework and guidance material that potential investors and government departments can refer to while working on developing PPP projects. The Government of Dubai has taken a lead while approving the first project ‘Dubai Supreme Courts Project’ on a PPP basis, it may be used to develop similar projects across other departments in the UAE and wider regions and will mark the beginning of a new era in developing infrastructure projects on Public Private Partnership basis.
References:
Law no. 22 ON THE ORGANIZATION OF PUBLIC-PRIVATE PARTNERSHIP IN THE EMIRATE OF DUBAI issued on 10/08/2015
**ABSTRACT:**

As part of the improvements performed by the United Arab Emirates Government in the infrastructure services, the Ministry of Infrastructure Development (MoID) decided to implement a Performance Based contract for the roads daily maintenance within the federal roads network.

The format of the contract was decided after a deep internal study and analysis within the Roads Department, which helped to create the best contract format for the MoID needs.

One main aspects to be evaluated was which items should be included within the performance criteria and the definition of their time response and level of service of each one of them as well as the works in which maintenance consist in.

The main goal of the project is to improve the roads maintenance quality and reduce the time response to attend those tasks. Some other services provided will help to reach this objective.

This contract is running from March 2017 and will be under implementation until March 2019.
1. INTRODUCTION

The Roads Department in the Ministry of Infrastructure Development (before Ministry of Public Works) is responsible for the Maintenance and Conservation of the federal roads network. The main objective of the department is to keep a good level in the roads maintenance since safety and operational conditions of the network depend on them and those are considered the main indicators for the Department to satisfy the citizens demand (safety and comfort).

As part of the Federal policies to increase quality and safety UAE government the national agenda 2021 was launched. This agenda includes a set of national indicators in different sectors as education, healthcare, economy, police and security, housing, infrastructure and government services. All of them are long-term, measuring performance outcomes according to each one of the national priorities comparing UAE with global benchmarks. The performance of these KPI values will be establishes by periodically monitoring.

In order to reach the goals contained in that National Agenda, UAE government needs to improve procedures and methods. One of these important changes came with the new format for the roads maintenance contract, evolving from the typical work orders to a first stage in a performance based contract.

Usually this type of contracts are awarded for minimum periods of 5 years, In this case and considering this is the first contract of this type applied in the Department (and the second one in the country for roads maintenance), internally is considered as a pilot project.

The contract started on March 2017, but to match the most common assignation criteria, the first stage will be implemented in a frame of time of 2 years and inside the terms of reference, it is considered the possibility to extend 2 more years and renew an extra one (depending on the contractor general performance during the contract). Works are divided in two main types: Performance or Routine Maintenance (RM) and Work Orders (WO).

The performance achieved until now is considered as highly satisfactory. MoID expectations are to extend during the extension (2019) the performance format to other elements that today are not contained in that criteria and keep the evolution to some other aspects currently in the scope of other contracts in 2020 (for example, pavements condition by indicators as IRI, Deflection and others).

2. BACKGROUND. MINISTRY OF INFRASTRUCTURE DEVELOPMENT NETWORK.

Maintenance and operation of the federal roads is responsibility of the Roads Department. The network which Ministry of Infrastructure Development is in charge of, comprises 750Km of center line, 1,250 Km of carriageway and 3,500Km lane.

The roads included in that network are the federal ones, within the Emirates of Sharjah, Ajman, Umm Al Quaim, Ras Al Khaima and Fujairah.
3. ROADS MAINTENANCE DESCRIPTION

Traditionally the roads maintenance in the Federal Roads has been performed by work orders, meaning that anytime something was needed it was requested to be fixed by the contractor through orders issued by the Department.

These work procedures were improved by introducing concepts as compulsory programming and delay penalties, but in any case the procedure still in the pure corrective maintenance side.

The traffic growth factors within the network indicating the users’ demand in the roads is large, and all factors are indicating that this tendency will remain as it is now at least for the coming 3 years.

Through the counting stations located in the roads network, some of those high growth factors detected, as an example were: 10% in road E-611 (Emirates road), or 8.5% in road E-88 (Sharjah – Dhaid – Masafi) from 2015 to 2016.

All these aspects are indicating a big increase in the roads use, and this aspect made the Ministry to search the way to reduce the time response and attention to incidents and accidents in the roads.

After studying different approaches, international experience showed that the best way to increase quality in services to road users is to do maintenance under performance criteria.

Some good examples and case studies were analyzed and all of them showed a performance increase in those parameters perceived by the conventional users as necessary to consider the road network as satisfactory (those parameters dealing with safety and comfort). Those examples showed also a good performance in the economy of the maintenance, due to the better administration and organization of times, teams, materials, etc.
4. CONTRACT FORMAT

As mentioned before, these type of contracts are designed to be awarded for a period from 5 to 10 years, but in this case as is the first experience in our administration for roads maintenance (and the second one in all United Arab Emirates) it was decided to do a pilot project for 2 years as first stage of a possible extensions of the same contract.

The TOR of this contract describes the mechanisms to analyze if the contractor performance during the contract and the criteria which will determine if is appropriate to renew it in a period of 2 + 1 years more.

One of the first tasks to be done by the Department is to define the scope of the works to be performed, and in what elements performance criteria can be applied (specially for the pavements case)

Regarding the pavements case, the Roads Department still keep the criteria that major maintenance (repair, rehabilitation or reconstruction) will relay in different projects with specific budgets related to awarded contracts (for example rehabilitation for 45Km in two carriageways in Sheikh Mohammed Bin Zayed) or reconstruction of asphalt in 35Km in Dhaid – Masafi road). As can be understood these type of projects will be executed under the umbrella of new contracts apart from the Performance Based one.

To define those elements that were going to be covered by the performance criteria, maintenance activities for the last 3 years were studied, defining which were the most common elements acting on and which ones had the biggest quantities.

A list of 12 elements to be controlled in performance was obtained. Then it was proceeded to create what in the contract are called TECHNICAL SPECIFICATIONS (TS).

TS are a compilation of different elements in which performance criteria will be applied. These TS contain some important aspects defining the elements, and the parameters in which performance will be calculated those points are:

- Scope
- Definition
- Level of service
- Response time
- Works to do
- Routine upgrading works (including the time to do the upgrading)
- Considered most common repair activities
- Performance criteria

TS can be then considered as the guideline for the performance works. All those activities are mentioned in the contract as RM or ROUTINE MAINTENANCE.
The final list of the Technical Specifications (elements) can be seen below:

<table>
<thead>
<tr>
<th>Code</th>
<th>Name</th>
</tr>
</thead>
<tbody>
<tr>
<td>TS00</td>
<td>CONTRACT</td>
</tr>
<tr>
<td>TS01</td>
<td>PAVEMENTS</td>
</tr>
<tr>
<td>TS02</td>
<td>SHOULDERS AND SIDE SLOPES</td>
</tr>
<tr>
<td>TS03</td>
<td>DRAINAGE STRUCTURES</td>
</tr>
<tr>
<td>TS04</td>
<td>SAND REMOVAL</td>
</tr>
<tr>
<td>TS05</td>
<td>FENCES AND CAMEL GRIDS</td>
</tr>
<tr>
<td>TS06</td>
<td>LIGHTING</td>
</tr>
<tr>
<td>TS07</td>
<td>ROAD MARKING</td>
</tr>
<tr>
<td>TS08</td>
<td>GUARDRAILS AND SAFETY BARRIERS</td>
</tr>
<tr>
<td>TS09</td>
<td>KERBSTONES</td>
</tr>
<tr>
<td>TS10</td>
<td>TRAFFIC SIGNS AND ROAD FURNITURE</td>
</tr>
<tr>
<td>TS11</td>
<td>VEGETATION CONTROL</td>
</tr>
<tr>
<td>TS12</td>
<td>GRAFFITI AND STICKERS REMOVAL</td>
</tr>
</tbody>
</table>

Figure 3. List of elements controlled by Performance within the contract

On the other hand, from the historical analysis mentioned before, those elements showing they had a low frequency of maintenance during the past years or their amount was not that big, will be fixed and maintained under a group called WO to WORK ORDERS. Works will be requested by a formal work order, which will have programs and budgets assigned to those specific works and represent only, the requested jobs.

As a highlight, within the RM group special attention was given to the Contract Management. This is considered one of the basic parts to be controlled within the Performance mode. Contract Management is represented by each one of the following parameters:
For those elements controlled by performance and having therefore a TS (Technical Specifications) there are two key factors for the contract success.

One is the Upgrading time, which is the time granted to the contractor to act in all elements controlled by performance, in order to reach the nominated level of service (contained in those TS). To reach a good level of service the contractor should provide a calendar with the works performed per section of each one of the roads in which MoID can identify at any time, what parts of network are still under the period assigned to the upgrading.
Once any element of the roads is upgraded, it is considered to be controlled by performance. In that case the calendars will be also the indicators in which the department will base the control for upgrading and performance (all programs should be approved by the MoID).

Once the elements are under performance control, time response is a key parameter to be controlled (that is the lapse of time passed since the failure notification is received and the contractor acts on it). Delays in this factor originated Non Conformity Reports that will be the main aspect to consider in the monthly penalty calculation. The other factor causing NCR is the quality of the performed works (if the quality is not complying with the specifications or the level of service is not the required one, also a NCR will be issued).

Below a table containing the upgrading and response time can be seen:

<table>
<thead>
<tr>
<th>Code</th>
<th>Name</th>
<th>ITEM</th>
<th>Upgrading</th>
<th>TIME RESPONSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>TS00</td>
<td>CONTRACT</td>
<td>Contract Management</td>
<td>Starting of the contract</td>
<td>24 months</td>
</tr>
<tr>
<td>TS01</td>
<td>PAVEMENTS</td>
<td>Pavements Potholes (more 30mm)</td>
<td>2 months</td>
<td>48 hours</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cracks pavements (&gt;20mm)</td>
<td></td>
<td>1 week</td>
</tr>
<tr>
<td>TS02</td>
<td>SHOULDER AND SIDE SLOPES</td>
<td>Edge break &gt; 100mm and rutting &gt;100mm</td>
<td>1 year</td>
<td>24 hours</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Potholes, pends 25mm crossing &gt;50mm and road marking</td>
<td></td>
<td>1 week</td>
</tr>
<tr>
<td>TS03</td>
<td>DRAINAGE STRUCTURES</td>
<td>Drainage structures, culverts, drains, shoulder cutout to 80% waterway</td>
<td>8 months</td>
<td>1 week</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Water channels &gt;80% clear</td>
<td></td>
<td>48 hours</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Manholes, curbs, concrete minimum 250mm</td>
<td></td>
<td>48 hours</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Pump stations working 50%</td>
<td></td>
<td>48 hours</td>
</tr>
<tr>
<td>TS04</td>
<td>SAND REMOVAL</td>
<td>Sand Removal (1cm height, and 30% coverage, 50cm shoulder coverage in kerbstone, 70cm shoulder in barrier, 5cm in footpath, height 100cm in fences or walls or 20% waterway culverts and drains)</td>
<td>6 months</td>
<td>2 hours</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Fences and walls 300cm and waterway and drains</td>
<td></td>
<td>24 hours</td>
</tr>
<tr>
<td>TS05</td>
<td>FENCES AND CAMEL GRIDS</td>
<td>Fences and gates closed all time</td>
<td>6 months</td>
<td>24 hours</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Camel grids operative all times and sand always 300mm below the top</td>
<td></td>
<td>1 week</td>
</tr>
<tr>
<td>TS06</td>
<td>LIGHTING</td>
<td>Solar panels clean</td>
<td>3 months</td>
<td>24 hours or permanent</td>
</tr>
<tr>
<td></td>
<td></td>
<td>85% lights working 100%</td>
<td></td>
<td>1 week</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Traffic lights operative 100%</td>
<td></td>
<td>2 hours</td>
</tr>
<tr>
<td>TS07</td>
<td>ROAD MARKING</td>
<td>Road Marking</td>
<td>2 years</td>
<td>7 days</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Other road markings (no regulatory)</td>
<td></td>
<td>1 month</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Catyes to keep legality</td>
<td></td>
<td>14 days</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>7 days</td>
</tr>
<tr>
<td>TS08</td>
<td>GUARDRAILS AND SAFETY BARRIERS</td>
<td>Guardrail and safety barriers</td>
<td>10 months</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Repair and repaint concrete barriers</td>
<td></td>
<td>7 days</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Permanent repairs to metallic guardrails</td>
<td></td>
<td>48</td>
</tr>
<tr>
<td>TS09</td>
<td>KERBSTONES</td>
<td>Kerbstones</td>
<td>1 year</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Permanent repairs</td>
<td></td>
<td>7 days</td>
</tr>
<tr>
<td>TS10</td>
<td>TRAFFIC SIGNS AND ROAD FURNITURE</td>
<td>Traffic signs and road furniture</td>
<td>1 year</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Re-paint traffic signs and signboard posts</td>
<td></td>
<td>72</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Permanent repairs to regulatory traffic signs</td>
<td></td>
<td>72</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Permanent repairs to other signs and furniture</td>
<td></td>
<td>30 days</td>
</tr>
<tr>
<td>TS11</td>
<td>VEGETATION CONTROL</td>
<td>Vegetation control</td>
<td>6 months</td>
<td>72</td>
</tr>
<tr>
<td>TS12</td>
<td>GRAFFITI AND STICKERS REMOVAL</td>
<td>Graffiti and stickers removal</td>
<td>6 months</td>
<td>7 days</td>
</tr>
</tbody>
</table>

Figure 5. Upgrading and response times for the TS elements

During the tender stage two main aspects were evaluated: technical capacity and knowledge of the contract. Both of them had associated the price of the contract, assigned to the Monthly Performance activities plus the price assigned to a predetermined items contained in the bill of quantities, to work with the Work Orders mode.

Financial wise the sum of performance works and work orders were considered as the awarded sum for the contract. The Performance works will be paid based on a monthly fixed lump sum appointed by the contractor in their proposal which was determined by the knowledge of the technical specifications, (specially the level of service the network needs to keep) and the knowledge of the network real situation.

The bill of quantities is based in the prices that the contractor gave to some activities (all of them with assumed assigned quantity, assuming that those quantities will be the ones executed during the contract. And based on that the sum of both concepts determined the economical proposal of the tenderers.

The time frame for the contract activities given to the contractors can be found below:
Figure 6. Time frame for the contract activities

<table>
<thead>
<tr>
<th>Phase</th>
<th>Duration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-Commencement</td>
<td>1.5 months</td>
</tr>
<tr>
<td>Project/Contractor Mobilization</td>
<td>1.5 months</td>
</tr>
<tr>
<td>Preparation of Contract Quality Plan (CQP)</td>
<td></td>
</tr>
<tr>
<td>Quality Control Plan (QCP)</td>
<td></td>
</tr>
<tr>
<td>Site Safety Plan (SSP)</td>
<td></td>
</tr>
<tr>
<td>Environmental Management Plan (EMP)</td>
<td></td>
</tr>
<tr>
<td>Traffic Management Plan (TMP)</td>
<td></td>
</tr>
<tr>
<td>Communications Management Plan (CMP)</td>
<td></td>
</tr>
<tr>
<td>Emergency Events Procedure (EEP)</td>
<td></td>
</tr>
<tr>
<td>Preparation and Training of MoID’s Road Inspectors</td>
<td>1 month</td>
</tr>
<tr>
<td>Preparation and Implementation of Protocols and Procedures</td>
<td>0.5 month</td>
</tr>
<tr>
<td>Preparation of Indicative Annual Program</td>
<td>1 month</td>
</tr>
<tr>
<td>Project Execution</td>
<td>24 months</td>
</tr>
<tr>
<td>Routine &amp; Maintenance Works as necessary</td>
<td>24 months</td>
</tr>
</tbody>
</table>

From the operational way the incidences for the road can be reported by 3 means, any user through an 800 telephonic line, the MoID inspectors and the contractor inspectors.

To record all incidences a special module in the already existing road assets management system (special IT application) was created. This module contain all the incidences registered in the road indicating the issued time, the element location, the response time, the person who reported, associated pictures, etc. This application can be installed in any device controlled by Android or IoS system.

All these incidences will be recorded in the system having three possible status: open, pending and closed. Open activities are those in which the contractor didn’t act still.

Pending will be those activities opened and that can’t be finalized on time due to special circumstances (extreme weather conditions or problems issuing the police permissions, for example). And close, all those activities that are already executed.

Figure 7. Control scheme through inspections
Figure 8. Mobile application screenshots

All data contained in the mobile application and the IT system (roads assets management) are synchronized and are able to show any person with access to the applications (Department’s personnel, inspectors, contractors), the following information:

- List of incidences (with any of the status indicated by colors: Open (yellow), Pending (red), and Closed (green)
- Map of those incidences
- The report of each one of them
- Control of the registered and authorized NOC (No Objection Certificates) issued by the Ministry for works performed in the roads

Registration can be done by using the module contained in the IT application too (operator from the 800 line or any person operating the system).

Figure 9. Registration of events and incidences in the system

5. CONTRACT EVALUATION

A fundamental task in order to know how the contract is performing is the evaluation of the contractor according with the terms of reference contained in the contract.

To control the contractor’s performance according with the contract criteria, the following procedure is applied:

1) Monthly inspections of the roads. The roads are divided in 3 types (A,B or C) depending on their category (Category A for Highways and Freeways, Category B for slow speed segments in Highways and type C for the arterial roads). For each one of them inspection will be performed in a minimum certain number of kilometers
which will represent the totality of the length of that type of road, and will penalize any no compliance for each element detected, in proportional way as if any defect is founded during the inspection (as described before by quality, level of service or failure in the time response).

Minimum inspection in Roads type A should is 40Km, Roads type B 20Km and Roads Type C 15Km. All of them have a certain weight in the system.

2) **Aleatory review of closed incidences registered within the system:** If any of those incidences is not closed in reality, will mean that it was never closed or as well the incidence was not closed on time after the registration and will create a no incidence

3) **Contract Management:** as mentioned before is an important part of the system in which documentation, data bases, accidents, environmental aspects are all analyzed and evaluated.

An example of the monthly evaluation for performance in the contract can be found below:

### ROUTINE WORKS

<table>
<thead>
<tr>
<th>Subtask</th>
<th>Task</th>
<th>SPI Penetration (%)</th>
<th>Detected Non-Compliance</th>
<th>Weighed Non-Compliance</th>
<th>Total of Weighed Non-Compliance</th>
<th>Final SPI Performance Achievement (Depends on the range of $80,000)</th>
<th>Applicable penalty</th>
</tr>
</thead>
<tbody>
<tr>
<td>T501</td>
<td>Pavement</td>
<td>100.00%</td>
<td>1.00</td>
<td>0.00</td>
<td>0.00</td>
<td>1.00</td>
<td>0.00</td>
</tr>
<tr>
<td>T502</td>
<td>Shoulder and Sidewalks</td>
<td>5.00%</td>
<td>1.00</td>
<td>0.00</td>
<td>0.00</td>
<td>1.00</td>
<td>0.00</td>
</tr>
<tr>
<td>T503</td>
<td>Drainage Structures</td>
<td>10.00%</td>
<td>4.00</td>
<td>0.00</td>
<td>0.00</td>
<td>4.00</td>
<td>0.00</td>
</tr>
<tr>
<td>T504</td>
<td>Guard Removal</td>
<td>10.00%</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>T505</td>
<td>Fences and Curb Glands</td>
<td>0.00%</td>
<td>22.00</td>
<td>0.00</td>
<td>0.00</td>
<td>22.00</td>
<td>0.00</td>
</tr>
<tr>
<td>T506</td>
<td>Lighting</td>
<td>5.00%</td>
<td>4.00</td>
<td>0.00</td>
<td>0.00</td>
<td>4.00</td>
<td>0.00</td>
</tr>
<tr>
<td>T507</td>
<td>Flood Marking</td>
<td>10.00%</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>T508</td>
<td>Guardrails and Safety Barriers</td>
<td>10.00%</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>T509</td>
<td>Kerbs</td>
<td>10.00%</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>T510</td>
<td>Traffic Signs and Road Furniture</td>
<td>10.00%</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>T511</td>
<td>Vegetation Control</td>
<td>5.00%</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>T512</td>
<td>Qualify &amp; Stickers Removal</td>
<td>5.00%</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

| SUBTOTAL | 100.00% | 95.00 | 74.50 | 25.50 |

Figure 10. Performance Works monthly control sheet
### Quality Control & Quality Assurance

<table>
<thead>
<tr>
<th>KPI Score</th>
<th>Monthly Score</th>
<th>3 Months Rolling Average Score</th>
<th>3 Months Rolling KPI Score</th>
<th>3 Months Rolling Average Score</th>
<th>Monthly Score</th>
<th>No. of NCR Unsolved On Time</th>
<th>Total No. of Reported NCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>#DIV/0!</td>
<td>#DIV/0!</td>
<td>#DIV/0!</td>
<td>#DIV/0!</td>
<td>#DIV/0!</td>
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<td>2.00</td>
</tr>
</tbody>
</table>

### Health and Safety

<table>
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<tr>
<th>KPI Score</th>
<th>Monthly Score</th>
<th>3 Months Rolling Average Score</th>
<th>3 Months Rolling KPI Score</th>
<th>3 Months Rolling Average Score</th>
<th>Monthly Score</th>
<th>No. of Accidents</th>
<th>Total No. Of Men Work Hours/10,000</th>
</tr>
</thead>
<tbody>
<tr>
<td>#DIV/0!</td>
<td>#DIV/0!</td>
<td>#DIV/0!</td>
<td>#DIV/0!</td>
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</tr>
</tbody>
</table>

### Traffic Management

<table>
<thead>
<tr>
<th>KPI Score</th>
<th>Monthly Score</th>
<th>3 Months Rolling Average Score</th>
<th>3 Months Rolling KPI Score</th>
<th>3 Months Rolling Average Score</th>
<th>Monthly Score</th>
<th>No. Of Accidents in Work Zones</th>
<th>Total No. Of Work Zones during the Month</th>
</tr>
</thead>
<tbody>
<tr>
<td>#DIV/0!</td>
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<td>#DIV/0!</td>
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<td>10.00</td>
</tr>
</tbody>
</table>

### Programming & Reporting

<table>
<thead>
<tr>
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<th>Monthly Score</th>
<th>3 Months Rolling Average Score</th>
<th>3 Months Rolling KPI Score</th>
<th>3 Months Rolling Average Score</th>
<th>Monthly Score</th>
<th>No. Of Accumulated Days of Delay</th>
<th>Total No. of Programmes &amp; Reports</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.75</td>
<td>1.36</td>
<td>2.95</td>
<td>2.75</td>
<td>2.20</td>
<td>1.00</td>
<td>11.00</td>
<td>4.00</td>
</tr>
</tbody>
</table>

### Database Management

<table>
<thead>
<tr>
<th>KPI Score</th>
<th>Monthly Score</th>
<th>3 Months Rolling Average Score</th>
<th>3 Months Rolling KPI Score</th>
<th>3 Months Rolling Average Score</th>
<th>Monthly Score</th>
<th>No. Of Accumulated Days of Delay</th>
<th>Total No. of Reports</th>
</tr>
</thead>
<tbody>
<tr>
<td>#DIV/0!</td>
<td>#DIV/0!</td>
<td>#DIV/0!</td>
<td>#DIV/0!</td>
<td>#DIV/0!</td>
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<td>0.00</td>
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</tbody>
</table>

### Emergency Works

<table>
<thead>
<tr>
<th>KPI Score</th>
<th>Monthly Score</th>
<th>3 Months Rolling Average Score</th>
<th>3 Months Rolling KPI Score</th>
<th>3 Months Rolling Average Score</th>
<th>Monthly Score</th>
<th>No. Of Emergencies Attended Out of Time</th>
<th>Total No. Of Call Out Emergencies</th>
</tr>
</thead>
<tbody>
<tr>
<td>#DIV/0!</td>
<td>#DIV/0!</td>
<td>#DIV/0!</td>
<td>#DIV/0!</td>
<td>#DIV/0!</td>
<td>#DIV/0!</td>
<td>0.00</td>
<td>10.00</td>
</tr>
</tbody>
</table>

### Customer Satisfaction

<table>
<thead>
<tr>
<th>KPI Score</th>
<th>Monthly Score</th>
<th>3 Months Rolling Average Score</th>
<th>3 Months Rolling KPI Score</th>
<th>3 Months Rolling Average Score</th>
<th>Monthly Score</th>
<th>No. Of Complaints Unsolved on Time</th>
<th>Total No. Of Complaints</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.80</td>
<td>0.80</td>
<td>0.80</td>
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<td>#DIV/0!</td>
<td>#DIV/0!</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Some extra services will be applied to the contract, as can be immediate attention to accidents whenever police or users require that service cleaning the road and returning the operational conditions again to the traffic.

Another service is the emergency attention. In this case the attention of the contractor will be focused in the instructions given by the corresponding authority (police or civil defence corp). The contractor should help in all situations to solve the requirements of the situation.

All equipment operating in the road will have the Ministry logo and the 800 line for incidences reports (in a 24/7 telephonic attention line) on where all incidences received will be registered, followed up and in case is not belonging to the Ministry scope, reallocated with the correct administration.

---

**Figure 11. Contract Management monthly control sheet**
6. PARTIAL RESULTS

During the time this contract has been active the results can be summarized in the following actions:

<table>
<thead>
<tr>
<th>ITEM</th>
<th>ACTION</th>
<th>QUANTITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pavements</td>
<td>Crack sealing and patching</td>
<td>750m²</td>
</tr>
<tr>
<td>Light poles</td>
<td>Rectification, removal, substitution, re-alignment, operation</td>
<td>840 pieces</td>
</tr>
<tr>
<td>Guardrail</td>
<td>Substitution, removal, rectification</td>
<td>8,400 m</td>
</tr>
<tr>
<td>Signboards</td>
<td>Change, rectification, removal re-alignment</td>
<td>800 pieces</td>
</tr>
<tr>
<td>Electric feeder pillars</td>
<td>Repair and connection</td>
<td>60 pieces</td>
</tr>
<tr>
<td>Edge fence</td>
<td>Re-alignment, replacement, fixation, removal</td>
<td>3,150 m</td>
</tr>
<tr>
<td>Camel grids</td>
<td>Cleaning, replacement, removal</td>
<td>25 pieces</td>
</tr>
<tr>
<td>General cleaning</td>
<td>Carriageway</td>
<td>45 Km</td>
</tr>
<tr>
<td>Sand removal</td>
<td>From carriageway and fences</td>
<td>6,100 m</td>
</tr>
<tr>
<td>Concrete barriers</td>
<td>Repair, re-alignment and removal</td>
<td>15 pieces</td>
</tr>
<tr>
<td>Plastic delineators</td>
<td>Replacement</td>
<td>38 pieces</td>
</tr>
<tr>
<td>Light cable</td>
<td>Removal and replacement</td>
<td>525 m</td>
</tr>
<tr>
<td>Emergencies</td>
<td>Removal of water accumulation</td>
<td>16,000 m³</td>
</tr>
<tr>
<td>Emergencies</td>
<td>Shoulders scouring repair</td>
<td>100 m³</td>
</tr>
</tbody>
</table>

And as per economic performance we can represent the last months as per the following control chart:
7. CURRENT AND FUTURE EXPECTATIONS FOR THIS CONTRACT

The most important activities for the rest of 2017 in this contract are the following ones:

- Road Marking (7000Km of road lines and 3500 road marks)
- Measurement of 2700 signboards retro-reflection)
- Culverts cleaning for storm prevention
- Physical inventory of all elements of the network considering useful information from the RAMS and the inspections for each one of these elements contained on it. As well determining the asset assessment condition index

Activities for next year (second year of contract) will be focused in:

- Maintenance of the levels of service
- Suggestions for improving the safety conditions (along road safety audits)
- Suggestions to improve the ITS network in the Ministry

The future conditions of these type of contracts are related to contain more elements within the performance format as for example:

- Pavements condition
- Bridge condition
- Slopes condition
- Protocols to improve accidents registration with the road safety audits to be performed

8. CONCLUSIONS

Performance Based Contracts are an efficient way to progress in roads maintenance. The decision to implement this type of contract is a step ahead in the management of the contracts in the MoID.

Not only because allows to relay the good conditions of the elements to be controlled and maintained in the company in charge, but, because this maintenance has to comply quality conditions, fast response and excellent services to the users, better attention to accidents and support for police and civil defence corps.
The improvement on the contract activities is now evident and the expectation for the economic development can be predicted as successful due to the partial results and evolution of the activities.

It is expected to renew this contact and implement more of this type for different scopes in the Ministry. Some of the most important technical conditions for the implementation of PPP contracts are now established at least for roads maintenance and from this contract structure some others can be written.
Benefits and challenges on PPP Contract Management for street lighting

<table>
<thead>
<tr>
<th>PAPER TITLE</th>
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</thead>
<tbody>
<tr>
<td>Benefits and challenges on PPP Contract Management for street lighting</td>
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</table>

<table>
<thead>
<tr>
<th>TRACK</th>
</tr>
</thead>
<tbody>
<tr>
<td>159</td>
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</table>

<table>
<thead>
<tr>
<th>AUTHOR (Capitalized Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ammar Majid SAFI</td>
<td>ELECTRICAL &amp; DEVELOPMENT ENGINEER</td>
<td>MINISTRY OF INFRASTRUCTURE DEVELOPMENT</td>
<td>UAE</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CO-AUTHOR(S) (Capitalized Family Name)</th>
<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shafia Ali ALKHEYAILI</td>
<td>CIVIL &amp; DEVELOPMENT ENGINEER</td>
<td>MINISTRY OF INFRASTRUCTURE DEVELOPMENT</td>
<td>UAE</td>
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</table>

<table>
<thead>
<tr>
<th>E-MAIL (for correspondence)</th>
</tr>
</thead>
<tbody>
<tr>
<td><a href="mailto:ammar.safi@moid.gov.ae">ammar.safi@moid.gov.ae</a></td>
</tr>
</tbody>
</table>

KEYWORDS:
Federal roads, energy saving, PPP, zero budget, CO2 emission, Smart Monitoring system, KPI's, Dimming curve, energy performance contract.

ABSTRACT:
The Ministry of Infrastructure Development is the executive arm of the UAE’s Federal Government, and has the responsible of the federal roads network projects execution, development, and maintenance for more than 750 Km of highways; containing more than 33,000 luminaries, falls under the MoID responsibilities.

During the development of this project various challenges have been appeared. These challenges effectively developed and upgrade the experience of the experts from looking to the best practice, bench marking with other countries, and sharing it with other entities to build new model in PPP contracts applicable with UAE federal rules and regulations. Therefore, every step is well studied, and all options to optimize the best solution are considered. Current challenges that have been raised within the upgrading projects have proven to affect directly the operation and maintenance of the Federal road network. As a result, the ministry has created new contractual mechanism that aims solely to reduce the expenses and increase both the sustainability and reliability of the maintenance process.

Recently, MOID has launched an initiative in which the environment is well protected and more than 50% energy saving of current electrical consumption (by using LED street light technology). The initiative has been studied thoroughly, including worldwide case studies, before it has been approved to be implemented in the form of PPP-Finance contract with no available budget (Zero Budget), by utilizing the savings from reducing the electricity consumption. This is the first contract of its kind to be launched federally.

Environmental wise, this contract will have a huge impact on the reduction of 11,888 tons CO2 emission, and elevating the serviceability conditions, which fulfills the one of the main objectives of MOID’s strategic KPI’s that is Energy Conservation.
One method to carry out the initiative and achieve the expected savings is to replace the conventional street lights system by LED. An advanced Smart Monitoring system, aiming to achieve all predefined KPI’s and dimming curve, leading to save an additional 23% of Energy consumption, and control the performance of this system.

The scope of work of this energy performance contract goes through various stages that are: design, supply, installation, operation, maintenance and finance, and 10 years payback a return period.
1 INTRODUCTION

Early 2014, UAE government launched the national agenda 2021 for UAE. The UAE National Agenda, which was developed by over 300 officials from 90 federal and local government entities, includes a set of national indicators in the sectors of education, healthcare, economy, police and security, housing, infrastructure and government services. These indicators are long-term, measure performance outcomes in each of the national priorities, and generally compare the UAE against global benchmarks. Government leadership to ensure their targets are achieved by 2021 periodically monitors the national indicators.

MoID Mission defined as "Achieving sustainable development in planning, establishing and maintaining infrastructure projects and organizing the national-housing sector through outstanding organizational performance according the world's highest standards", and the Vision is "International pioneering in constructing infrastructure projects for a country that is second to none".

One of the Ministry of Infrastructure Development (MoID) initiatives is "Energy Performance Contract (Retrofit program) for replacement the Federal street lighting by LED System to save the power consumption (Design, Replace, Operation, Maintenance & Finance) " Project to support the UAE National Agenda1, achieve sustainability KPI's, and to enhance public-private partnership (PPP) at the federal level. This case study has been applied into the United Arab Emirates (UAE) federal road network (Northern Emirates), covering more than 750KM with different types of roads' classification as shown in table (1-a).

<table>
<thead>
<tr>
<th>Sr. ROAD NO.</th>
<th>ROAD NAME</th>
<th>LENGTH (Km)</th>
<th>CLASS</th>
<th>date of maintenance</th>
<th>No. of lanes</th>
<th>speed</th>
<th>poles height</th>
<th>Total NO. of lumaire</th>
<th>TOTAL no. OF POLES</th>
<th>distance between poles</th>
<th>power wattage</th>
<th>future cross-section</th>
</tr>
</thead>
<tbody>
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<td>48</td>
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<td>2007</td>
<td>0</td>
<td>120</td>
<td>14</td>
<td>4000</td>
<td>1200</td>
<td>50-55</td>
<td>450</td>
<td>4 lanes</td>
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<tr>
<td>2 E16</td>
<td>MANAMA - BRAM ROAD</td>
<td>98</td>
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<td>2003-2005-2006-2008</td>
<td>2</td>
<td>120</td>
<td>14</td>
<td>5172</td>
<td>1764</td>
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<tr>
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<td>DIBA - SHARJAH - DHAID ROAD</td>
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<tr>
<td>4 E21</td>
<td>DHAID - KHAJA - AL MAJID ROAD</td>
<td>31</td>
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<tr>
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<td>TOWYN - SHBA ROAD</td>
<td>36</td>
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<td>2009-2012</td>
<td>2</td>
<td>120</td>
<td>14</td>
<td>564</td>
<td>485</td>
<td>55-60</td>
<td>620</td>
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<tr>
<td>7 E66</td>
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<td>PRIMARY ARTERIAL</td>
<td>2002</td>
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<td>120</td>
<td>14</td>
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<td>624</td>
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<tr>
<td>9 E86</td>
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<td>170</td>
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<td>55-60</td>
<td>250</td>
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<tr>
<td>13 E66</td>
<td>KUHAFAN - DIBA ROAD</td>
<td>34</td>
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<td>2005-2012</td>
<td>1-2</td>
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</tr>
<tr>
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<td>230</td>
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<td>3</td>
</tr>
<tr>
<td>15 E66</td>
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<td>14</td>
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<td>55-60</td>
<td>250</td>
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</tr>
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<td>16 E66</td>
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<td>34</td>
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<td>1-2</td>
<td>120</td>
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</tr>
<tr>
<td>17 E66</td>
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<td>2005-2012</td>
<td>1-2</td>
<td>120</td>
<td>14</td>
<td>230</td>
<td>125</td>
<td>55-60</td>
<td>250</td>
<td>3</td>
</tr>
<tr>
<td>18 E66</td>
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<td>34</td>
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<td>1-2</td>
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<td>14</td>
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<td>125</td>
<td>55-60</td>
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</tr>
<tr>
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<td>230</td>
<td>125</td>
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</tr>
<tr>
<td>20 E66</td>
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<td>125</td>
<td>55-60</td>
<td>250</td>
<td>3</td>
</tr>
<tr>
<td>22 E66</td>
<td>KUHAFAN - DIBA ROAD</td>
<td>34</td>
<td>PRIMARY ARTERIAL</td>
<td>2005-2012</td>
<td>1-2</td>
<td>120</td>
<td>14</td>
<td>230</td>
<td>125</td>
<td>55-60</td>
<td>250</td>
<td>3</td>
</tr>
</tbody>
</table>

Table (1-a): UAE Federal roads’ classification

Previously MOID Used to build the infrastructure through out the conventional contract mainly, the payments and cash flow depends on the execution percentage without considering any performance or KPI's impact. This pioneer project with all challenges appeared during the implementation stages will reflect effectively the modification in the UAE law of energy conservation and construction field for future smooth and dynamic PPP contracts creation.
2. PROJECT LIFECYCLE

Public-Private Partnership projects lifecycle and mechanism vary from country to another and depends on rules, regulations, laws, governments needs, and common practice inside the country. This project is one of the most complex mechanisms in its financial parameters, where it includes the return on investment that is related to energy conservation. Therefore, lifecycle has many important milestones that we will focus on in this case study.

STAGE 1: DATA COLLECTION.

Strong study and right decision making are depends on accurate information. In early 2015 MOID started collecting the power consumption, assets, and technical information in details. The main challenges here are the accuracy, lack of information in old-existing roads, and the solution was to do site visits for all old-existing roads to verify the date of construction, pole height, distance between poles, road speed, luminary wattages, number of luminaire per poles, number of poles in each street, and road categories as shown above in table (1-a)

STAGE 2: FEASIBILITY STUDY

A- TECHNICALLY ACHIEVED:

In the data collection stage of all roads configurations, we started the technical studies for the federal road network. Meanwhile, we look at the current lighting level, uniformity, and glare. In sequence, the team has started simulating lighting parameters by replacing the HPS lamp to LED luminary. Parallel to that the lighting level reduced while using LED in the study to gain the optimal exploitation of the LED advantages “Better Uniformity than HPS”. High uniformity phenomena in LED directly effected the power consumption positively by increasing the percentage of electricity saving. The main challenge here in road configurations of high mast poles (16-25) meters, long distance between pole reached up to 120 meters, and express way category with 3-5 lanes additional to this the limitation in LED chip in the roads Shk. Mohammad bin Zayied Road “E311”, Shk. Khalifa Road “E84” & Emirates Road “E611”. Those roads technically failed to achieve both lighting parameters and power saving, and excluded from phase one study.

B- COMMERCIALY FEASABLE:

Optimal retrofit design solution is the main key factor for our successful feasibility study (Table 2-a). The design study effected the average of the future power consumption, energy saving, and feasibility of retrofit roads. The current tariff KW/H in Dirhams, current HPS Lamp power consumption the average price of the LED luminary, installation cost, and the current maintenance cost per HPS luminaire approximately 30$. All these factors reflected the financial feasibility study. Our recommendations to apply phase one of retrofit program as the following:

1- Apply the program on (550 KM that has (12 – 14m) poles height, (40-60) meters poles spacing, (250-400) watts HPS lamp power) only.

2- Payback with 8.5 years (without lighting management system, without applying dimming function, and based on subsidized tariff rate).
### Table (2-a): Summary of Feasibility study

<table>
<thead>
<tr>
<th>Zone</th>
<th>nr</th>
<th>Km</th>
<th>nr</th>
<th>Total number of luminaire</th>
<th>Power consumption</th>
<th>Amount of power</th>
<th>Cost</th>
<th>Project period</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7</td>
<td>275</td>
<td>11856</td>
<td>14475</td>
<td>46,011,024.00</td>
<td>9,202,204.00</td>
<td>4,985,210.88</td>
<td>1,495,563.26</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>230</td>
<td>11856</td>
<td>14475</td>
<td>6,120,672.00</td>
<td>2,738,901.60</td>
<td>669,081.00</td>
<td>585.13</td>
</tr>
<tr>
<td>Total</td>
<td>11</td>
<td>505</td>
<td>11856</td>
<td>14475</td>
<td>46,011,024.00</td>
<td>9,202,204.00</td>
<td>14,114,882.88</td>
<td>4,234,464.86</td>
</tr>
</tbody>
</table>

### Map 1: UAE Federal Roads Network and scoop of work project
STAGE 3: BEST PRACTICE COMPARISION:

Searching for the Best practice and global benchmarks in PPP contracts, the best start is to build the right contract structure and align government needs up to the target along with developing the resource experiences. MOID studied different practices in PPP energy performance contracts and Financing as shown in table 3-a. The challenges that MOID team faced during the study were:

- It was the first practice in the region, and few available practices internationally.
- LED technology is still new.

<table>
<thead>
<tr>
<th>Payment schedule</th>
<th>USA</th>
<th>Austria</th>
<th>Spain</th>
<th>Proposal for MoID</th>
</tr>
</thead>
<tbody>
<tr>
<td>Down payment</td>
<td>DOWN payment</td>
<td>Fixed payments (extra savings are shared; shortfall in savings are compensated through a bond)</td>
<td>Fixed payments (no implication for over or underperformance of the products)</td>
<td>Lighting company is paid upfront and guarantees savings with bonds during the payback period (extra savings are kept by the customer; shortfall in savings are compensated through a bond)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Payback period</th>
<th>USA</th>
<th>Austria</th>
<th>Spain</th>
<th>Proposal for MoID</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solution price exceeded savings over the guarantee period (10 years), a balloon payment has been made for the difference between solution price and savings (equivalent to down-payment concept)</td>
<td>Payment fully recovered from savings during contract period</td>
<td>Bonds are issues for the contract / payback period</td>
<td>Austria</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Audit details</th>
<th>USA</th>
<th>Austria</th>
<th>Spain</th>
<th>Proposal for MoID</th>
</tr>
</thead>
<tbody>
<tr>
<td>Metering systems installed (Tele-management system for outdoor; alternative – Indoor)</td>
<td>Sample-based M&amp;V plan</td>
<td>Sample-based M&amp;V plan</td>
<td>Sample-based M&amp;V plan &amp; each 6 months</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Maintenance and repair</th>
<th>USA</th>
<th>Austria</th>
<th>Spain</th>
<th>Proposal for MoID</th>
</tr>
</thead>
<tbody>
<tr>
<td>Customer is to provide access to site; ESCO is to resolve the issue within the agreed SLA and KPI’s</td>
<td>Customer is to provide access to site; ESCO is to resolve the issue within the agreed SLA and KPI’s</td>
<td>Customer is to provide access to site; ESCO is to resolve the issue within the agreed SLA and KPI’s</td>
<td>Any</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Performance guarantees (lux, energy savings)</th>
<th>USA</th>
<th>Austria</th>
<th>Spain</th>
<th>Proposal for MoID</th>
</tr>
</thead>
<tbody>
<tr>
<td>Light level, energy savings</td>
<td>Light level, energy savings</td>
<td>Light level, energy savings</td>
<td>Any</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Guarantees</th>
<th>USA</th>
<th>Austria</th>
<th>Spain</th>
<th>Proposal for MoID</th>
</tr>
</thead>
<tbody>
<tr>
<td>ESCO takes the risk for energy savings</td>
<td>ESCO takes the risk for energy savings</td>
<td>ESCO takes the risk for energy savings</td>
<td>Any</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Handover process (partial/completely)</th>
<th>USA</th>
<th>Austria</th>
<th>Spain</th>
<th>Proposal for MoID</th>
</tr>
</thead>
<tbody>
<tr>
<td>Partial, per phase</td>
<td>Partial, per phase</td>
<td>Varies per project</td>
<td>Complete</td>
<td></td>
</tr>
</tbody>
</table>

Table (3-a): PRACTICE COMPARISION
STAGE 4: GOVERNMENT APPROVALS AND CORDENATION:

UAE federal rules and regulation are very organized therefor any new initiative is monitored and follow up by top management, in sequence any initiative needs the cooperation between several different entity needs the cabinet and prime minister approval in October 2015 the full study and recommendation have been raised to the cabinet for approval. And in November MOID got the initial approval and early January 2016 got final approval. The main challenge was how to modify the regulations and federal procedures between MOID, ministry of finance and, federal electrical and water authority. The best solution was to consider this initiative as a pilot project and records all notes and new regulation needs in full report during the implementation and execution and higher it up to modify the procedure, process, and regulations to be more homogeneity and integrated with UAE rules.

STAGE 5: PRE-QUALIFICATION:

A - PURPOSE

Ministry of infrastructure development (hereinafter referred to as “the Employer”) is seeking proposals from interested Energy Services Companies (ESCOs) that are capable of providing comprehensive energy audits, capital improvement installation services under a turnkey approach and project management services to upgrade the lighting for the street lighting of the federal roads. The Employer’s objective in issuing this Request is to enter into a guaranteed performance-based energy efficiency contract with a qualified ESCO, which will focus on energy cost reductions (hereinafter referred to as “energy savings”).

On the other hand the prequalification stage was important to MoID to filter the participants for round table discussions and keep the tender evaluation depends only on commercial offer.

B - Method of technical evaluation

Chart (5-a): Technical Evaluation Criteria
As shown above in the diagram, pre-qualification are divided into three categories:

1- **Design** submittal is mainly concern about achieving the 50% minimum energy saving and lighting design level, taking in consideration the network upgrading and development within next 15 years. These two achievements were mandatory with zero weight (either pass OR fail) for all consortiums. The weight of evaluation divided to: 40% for consortiums and 60% for products. The consortiums must achieve minimum 70% to pass.

2- ** Consortiums:**

   This category is mainly focusing on:

   a- Companies in consortiums: The companies criteria’s were divided into legal structure, financial strength for all members, organization, consortium profile, work load and resources, management system, ISO accreditation, relevant experience, previous experience in UAE as street light, and maintenance work, R&D Center, number of patented, and accredited laboratories.

   b- Project organization: The project organization criteria were divided into progress of work plan, work methodology, and project team structure that includes the supervision staff. Additional to these consortiums, members must provide audited financial statement for the last 3 years. After evaluating the consortiums, they presented about their experience and expertise in energy efficiency project worldwide, experience and expertise in measurement and verification M&V, and demonstrated their capacity to verify and reconcile energy savings under a guaranteed saving energy performance contract in related sector.

3- **Products:** Long-term relation between public and private sector (more than 12 years) has big liability and risk mainly if we take the energy and lighting performance as main factors considering the circumstances and weather conditions in gulf countries during the summer. Therefore, product evaluation is important too in our project. MoID evaluate the reliability of products by checking several parameters in luminaries (LED chip, mechanical build quality, electrical build quality, maintenance and life expectancy, photometric performance and thermal management, and driver), and smart public lighting management system "SPLMS" to insure the compatibility and durability of the products during the operation stage.
C - Result of prequalification:

![Chart showing requests received, met minimum requirements, and round table discussion & bedder participants]

Table (5-b): Pre-Qualification Results

STAGE 6: BUILDING CONTRACT (ROUNDTABLE DISCUSSION – WORKSHOPS – INTERNAL DISCUSSION):

Cooperation and participation in this project between public and private sectors are important and essential to achieve the target successfully. Within six months a regular weekly meetings have been done to discuss the critical points and align the vision. During these workshops, many important decisions have been taken as shown in the table (5-a) below:

<table>
<thead>
<tr>
<th>Sr.</th>
<th>Point For discussion</th>
<th>Solution</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Project Duration</td>
<td>1.5 years Execution, 10 years operation and maintenance and 1 year defect liability</td>
</tr>
<tr>
<td>2</td>
<td>Operation hours</td>
<td>12 Hours, as a worst-case scenario during the winter. (It affect the payback if it is more).</td>
</tr>
<tr>
<td>3</td>
<td>Tariff rate</td>
<td>0-10,000 KWH is 0.28 AED, and above 10,000 KWH is 0.43 AED. In case any increase in tariff MoID will pay the extra difference, and if decreased contractor will revise the energy bill payment according to the difference</td>
</tr>
</tbody>
</table>
| 4   | Method of payments              | • Zero payments during the execution stage and the payments start at the operation and maintenance stage with 10 years duration of total 120 payments.  
• The payment start quarterly in the execution stage (6 payments during 18 months), and monthly during the operation and maintenance stage with 120 payment during 10 years of total 126 payments.  
Upon the consortiums and banks requests to be able to finance the project. MoID agreed with the consortiums to choose between the methods of payments either |
| 5   | Testing and commissioning       | New abbreviations created:                                                                   |
|     | procedure                       | • Partially handover: During the execution stage, MoID will inspect each road separately.  
• Initial handover: During operation and maintenance stage each 6-month the regular inspection.  
• Final handover: after the complete the operation and maintenance stage and also defect liability period |
| 6   | Electricity payment mechanism    | Contractor will pay the electricity bill and then claimed to MoID                            |
| 7   | Breakdown cost                  | Total value hereof (lump sum) including all guarantees, warrantees and other requirements included herein:  
1- Analyzing available information, filed inspection and preparing design for new lighting and, controlling systems in order to achieve electrical consumption reduction percentage stated in the contract.  
2- Supply new LED luminaries with controlling system with the server and necessary equipment’s as per MoID specification.  
3- Removing current lighting system and replace them by new lighting and controlling system in addition to traffic diversion cost and providing safety and security factor.  
4- Performing all test and commission upon completion, performing trial operating and obtaining a partial preliminary handing over certificate.  
5- Operation and maintenance of lighting system for 1 years starting from the date of passing completion tests and complete all roads subject to contract. The price include but not limited to energy consumption for beneficiary authorities, replace all defected luminaries and devices, apply and, pass the periodicals tests and commission.  
6- Performing final tests in addition to any repairs in order to issue performance
10% for each payment and return back when the contractor passes the Initial handover test at the end of the year.

Additional to this MoID clearly defined the scope of work as the following section:

**Section 1.** Design, Supply, Finance, Installation, test and commission.

**Section 2.** Operation & Maintenance for all installation done by the contractor. Same as

**Section 3.** Handing over.

NOTES: any components Or materials installed by Awarded contractor. It will be his responsibility during the whole period (execution, Operation & maintenance).

**Section 1.** Design, Supply, Finance Installation, test and commission of LED luminaire (Including LED, Driver and fixture, Tele management system and all accessories and server):

- Bidder has to design, supply, install, test and commission the LED lights within 18 months from the date of award. Bidders are requested to bid only if they will be able to supply, install, test and commission the tendered quantity of streetlights on site within a period not exceeding 18 months from the date of placement of letter of award to successful bidder.

- Bidder has to deliver the de-installed old light luminaires/lamps on as is basis to an indicated storage facility; the location will be in U.A.E and will be informed by ministry.

- Successful bidder has to return of the old luminaire and fixtures to the MOID and collect the verified quantity

- The specifications of the LED luminaries shall be as per given in the Technical Specifications. And compline with MOID specification as per volume 3.

- All the machineries and instruments required for the implementation of the project is to be arranged by bidder and their expense is also to be borne by them, MOID will only provide the coordination and helping the contractor to obtain all the permits access in due time when it needed.
• Storage for dismantled luminaires/lamps would be in the scope of work of awarded contractor.
• Secured temporary Storage for new luminaires/lamps will be provided by Bidder.
• Bidder shall ensure proper recording of the dismantled conventional fixtures and on daily basis verification certification has to be taken from MOID to avoid any discrepancy at a later stage.
• Bidder shall have to collect sign-off of the luminaires dismantled and prepare detailed inventory of the LED installation in the form of Road-wise report to be submitted to MOID.
• Bidder to undertake marking of poles (pole numbering) for each LED new Luminaires installed along with pole location (GPS Coordinate) and electrical cabinets’ point details in the tele – management system.
• Bidder will implement all Infrastructure Assets in Operations and Maintenance Asset Management platform and installed in MOID building with all needs accessory.
• Luminaire casing to also have a unique serial number (e.g. on a sticker). Vendors to submit the weekly report of installations, giving details of switch point, pole GPS Coordinates and the corresponding Luminaire serial number.
• Bidder shall ensure provision to include appropriate local pole cabling, phase wire work, fuse box, surge protector, earthing arrangements, clamps, nut-bolts, brackets and arms etc, wherever necessary for LED installations outside of major Infrastructure refurbishment requirements (rotten poles, underground cabling and feeder pillar) and it will be consider as work order after MOID approval the coordination will be mutual approve to verify the process.
• Site survey has been arranged and tender will be used the baseline signed off by all the bidders.
• The inventory details provided are tentative, however the supply shall be as below:
  - Road wise survey shall be conducted by the vendor to assess actual installations required before releasing supplies and shall also submit such report to MOID and take approval thereof.
  - The supply shall be governed by the survey report.

Section 2. Operation, Repair & Maintenance of LED fixture (Include, LED, Driver and fixture, Tele management system, all accessories and, server):

Scope of work Under Warranty:

• The LED luminaries, mechanical structures, electrical works including overall workmanship of the LED street-lighting system must be warranted against any manufacturing defect for the project period of 10 years as specified in the RFP.
• MOID is operating the system in terms on the burning hours; ESCO is having full access to monitor burning hours only.
• During the project period of 10 years, following maintenance will be required to be carried out by the bidder:
  o Mechanism for identification of faulty fixtures and lodging and monitoring of complaints. It is imperative that a call center will be in place for handling complaints and emergency calls.
  o Repairing / replacement of all defective components and sub-components of the system as per the requirement to ensure proper operation/functioning of the system.
  o The scope of work includes repairing/replacement of part(s) /system to make the system functional within warranty period whenever a defect is noticed or reported at site as per SLA’s below.
  o Secured storage facilities for spares is to be provided by awarded contractor.
• Bidder shall perform the following activities under maintenance service:
  o Except in case of Energy Supply related issues, the bidder will notify MOID who will then coordinate with the Concerned Utility and get the correct reason and repair for failure recorded.
  o Attend complaints passed on by through their customer complaint mechanism (including call center from MOID)
  o In case any issue is not related to the scope of work of the bidder but has consequences on lighting, a bidder representative is to be notified and has a right to participate in the discussion on corrective measures.
• The bidder shall ensure that all the SLA’s are in line with the following otherwise the penalties below are applicable.

STAGE 7: FIELD MEASUREMENTS

- ENERGY VERIFICATION & ASSETS INVENTORY: energy bill verification is the backbone in such type of contract. Mainly all budget Energy saving KPI’s, liability and, payments depend on Energy consumption and, bill amounts therefor, during the round table decision MoID and, qualified consortiums agreed to establish for joining survey team to verify more than 50% of the streetlight phase one and, 100% for assets inventory in roads network. The team measures and collect the following data as shown in forms a, b and c.

**General Details**

<table>
<thead>
<tr>
<th>Road Name</th>
<th>MANAMA – Quasadat – sham Aldoraa</th>
<th>Road Number</th>
<th>E16</th>
<th>Date of Road Establishment/Maintenance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Road Description</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Road Length</td>
<td>937</td>
<td>Type</td>
<td>PRIMARY ART TICAL</td>
<td>Reference lighting level</td>
</tr>
<tr>
<td>Speed Limits</td>
<td>100-120</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Roads Location "coordinates":**

<table>
<thead>
<tr>
<th>Starting Point</th>
<th>Ending Point</th>
</tr>
</thead>
<tbody>
<tr>
<td>E° N°</td>
<td>E° N°</td>
</tr>
<tr>
<td>2802491.05m</td>
<td>387230.47m</td>
</tr>
<tr>
<td>2976537.90m</td>
<td>2785357.90m</td>
</tr>
</tbody>
</table>

**Other Notes:** Starting from interchange no. 5

**Lighting Details:**

<table>
<thead>
<tr>
<th>Pole Height</th>
<th>Distance Between poles</th>
<th>No. of Luminaire per pole</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>55-60</td>
<td>2PL NO#001-722 4 PL NO# 726-1784</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Total Number of Poles</th>
<th>Total Number Of Luminaire</th>
<th>Total No. of Feeder pillars</th>
</tr>
</thead>
<tbody>
<tr>
<td>1784</td>
<td>5172</td>
<td></td>
</tr>
</tbody>
</table>

Luminaries Power: 250-400 Future Cross-section 'a.d.ditional number of teams'

**Reference lighting level**

<table>
<thead>
<tr>
<th>Category</th>
<th>Road coating luminance</th>
<th>Disability glare T180 max (+5% for low luminance sources) %</th>
<th>Proximity lighting SR 2 min. (in absence of traffic zone with proper requirements near the lane)</th>
<th>Maintenance Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>ME2</td>
<td>1.5 [cd/m2] 0.4 [cd/s]</td>
<td>0.7</td>
<td>0.5</td>
<td>0.8</td>
</tr>
</tbody>
</table>

Form (a)
### A. Feeder pillar details

<table>
<thead>
<tr>
<th>S.N</th>
<th>Pole height</th>
<th>Total No. of poles / F.P</th>
<th>No. of luminaires / pole</th>
<th>Total No. of luminaires</th>
<th>Total No. of working luminaires during survey</th>
<th>Luminaires wattage</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>12</td>
<td>24</td>
<td>4</td>
<td>96</td>
<td>89</td>
<td>0.25</td>
</tr>
</tbody>
</table>

### B. Power Measurements details

<table>
<thead>
<tr>
<th>S.N</th>
<th>Phase</th>
<th>Amp/phase</th>
<th>Voltage /phase</th>
<th>K watts/phase</th>
<th>Total measured KW</th>
<th>Total measured KVA</th>
<th>Total Power factor</th>
<th>Total Calculated KW (Theoretical)</th>
<th>Total losses Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>RED</td>
<td>48.50</td>
<td>236.90</td>
<td>9.76</td>
<td>28.80</td>
<td>34.60</td>
<td>0.85</td>
<td>22.25</td>
<td>29%</td>
</tr>
<tr>
<td>2</td>
<td>Yellow</td>
<td>54.30</td>
<td>231.80</td>
<td>11.37</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Blue</td>
<td>36.50</td>
<td>239.90</td>
<td>7.70</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Neutral to ground</td>
<td>7.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Megger value (Y/G earth)</td>
<td>V.High</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### C. Feeder pillar Conditions

<table>
<thead>
<tr>
<th>S.N</th>
<th>Foundation</th>
<th>Enclosure</th>
<th>Components</th>
<th>Issue: Problems (phase loss/jumper/ floating neutral/no earthing/breaker/high voltage...etc)</th>
<th>Other connected loads ( Radar / Advs ...etc)</th>
<th>Others</th>
<th>A/F P must be replaced during installation to be replaced.</th>
<th>C- No Need</th>
<th>B - recommended</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Good</td>
<td>Good</td>
<td>Good</td>
<td>Floating Neutral</td>
<td></td>
<td></td>
<td></td>
<td>C</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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Note: FEWA tariff slices for government from 0-10,000 KWH monthly = 28 fils and, from 10,000 and above = 43fils
Form (c)

A- TRAFFIC FLOW COUNTS AND DIMMING CURVE:

- MOID target in this project is to gain the maximum energy saving, retain on investment and, environmental impact with deliver to the driver same SLA’s or better. After study and analysis for the traffic flow data in MoID roads network and, applying the standards to select optimal lighting level.
- Using more than 25 Automatic vehicle counting stations (Map 2) installed among the federal roads with analyzing Data collection and obtains the yearly average traffic flow hour by hour And, classify the data collection into (LV OR HV) example shown in table 7-a & 7-b. This data allow MoID forecasting / projection the lighting level for the next year and, apply dimming curve during the less traffic demand, with grate saving in energy consumption more than 23%.
Table (7-a) - Sample of data analysis for E88 Road.

Table (7-b) - Sample of Data Analysis for E88 Road.
STAGE 8: FINALIZING THE CONTRACT:

After completing the technical and contractual data MoID team prepares the contract document and it as divided into 4 Volumes as shown in table 8-a

<table>
<thead>
<tr>
<th>Sr.</th>
<th>Volume No.</th>
<th>Document</th>
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<td>- Contract</td>
<td>- Contract Agreement</td>
<td>- Letter Of Acceptance</td>
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<td>- Method of payment</td>
<td>- Price break down</td>
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<td>- Bed bond</td>
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<td>General &amp; Particular Conditions</td>
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<td>- Particular Conditions “A&amp;B”</td>
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<td>III</td>
<td>Employer Requirements</td>
<td>General Specifications</td>
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<td></td>
<td></td>
<td>Street Light Technical Details Specifications</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>• Introduction</td>
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<td>• Scope Of Works</td>
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<td></td>
<td>• Street Lighting Standards Of Comfort &amp; Testing And Commissioning</td>
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<td>• KPI’s</td>
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<td>• Warranties</td>
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<td>• Trainings</td>
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<td>• Luminaire Performance Requirements</td>
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<td>• Bill Of Quantity</td>
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<td>• Energy Saving Requirements</td>
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<td>• Base Line Equation &amp; Energy Consumption</td>
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<td>• Flow Chart Of Operation</td>
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<td>• Work Methodology Requirements</td>
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<td>• Index And Question Addendum</td>
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</table>

Table (8-a) : Contract Documents

STAGE 9: TENDERING AND AWARDING:

For more transparency awarded consortium criteria simple by the lowest price,

STAGE 10: CONTRACT MECHANISM

A- EXECUTION:

After awarding this project to the consortium the following procedures to be consider:

1- During the mobilization period 30 days the contractor will provide:

1.1 Staff CV’s for approval from MoID
1.2 Material submittal to be provide to MoID for approval
1.3 Safety plan and methodology of works
1.4 Work progress
2- MOID will hand over the roads one by one to the consortium with joint survey the assets damage, where the energy verification to be with Federal Water and Electrical Authority to insure the accuracy of the energy meter.

3- Consortium should submit the design for review and approval after the material approval.

4- Consortium delivers to the site the material for inspection and start replacing the old luminaries.

B- HANDOVER & TESTING AND COMMISSIONING:

Each road after replacing all luminaries will inspect from MoID and consortium for approval during this stage MoID team will check and verify the energy consumption, lighting level, lighting parameters and, energy saving to compare them with the required level. In case one of these parameters or all of them not achieved the contractor has 2 month to fix it before applied fines during the execution stage.

C- OPERATION & MAINTENANCE:

During this stage the MoID responsibility is limited to:

- Monitor the performance, time response and, service level agreement delivered from consortium during the 10 years.
- Each 6-month MoID team and consortium has to inspect the roads and verify the roads conditions to insure that all KPIs are achieved.
- Review and check the monthly payment and energy bill in case any deviation in the amount of energy consumption to verify it.
- Applied fine required and Payment of Retention money 10% to the consortium at the first month from the next year.

D- FINAL HANDOVER

After consortium complete the contractual period (10 years) MoID team and consortium will inspect the roads as a routine procedure and insure all parameters level are correct and then after completing the handover from the consortium to MoID the Operation and maintenance will be a part of MoID responsibility.

CONCLUSION: BENEFITS AND CHALLENGES

We live in an age where energy is an absolute necessity to maintain our lifestyle and it goes beyond the basics of providing comfort and convenience. We only need to think back to the last power cut to understand the reality of life without energy, when nothing that you’ve come to be dependent on works.

- Saving energy means wasting less money
- Saving energy means wasting less primary fuel (coal, gas, oil, uranium)
- Saving energy means producing less pollution
- No accurate Energy bill
- Existing Advertising boards
- First experience with PPP projects in MOID, need to elaborate and contribute with existing rules and regulations.
- Few experiences for PPP projects in UAE
- On site energy verification for more than 50% of the Roads Network
- Issue the Executive Roads Order
- Cooperate with POM, MOF & FEWA for new mechanism.
- Benchmarking with other countries, Workshops & Round Table discussion with the consortiums
- Energy Consumption 30,844,047.75 KWH/Year
- Pay back: 8.5 – 9.5 Years
- Today Energy Cost 9,253,374.87 AED/Year
- Energy Saving 53-65 % & with dimming 75-85% Energy saving
• 11,888.84 metric tons CO2 Reduction.
• “ENERGY SAVINGS ARE REAL” – As the LEDs improve, and the manufacturers develop the technology, the energy savings are being realized and continue to increase.
• “YOU CAN IMPROVE VISIBILITY & Safety” – The change from HPS to white light have improved visibilities.
**References:**


Blockchain and Toll Collection: A Unified Tolling Network

Matthew MILLIGAN
Managing Partner
Milligan Partners
USA

Tolling, transit, parking, blockchain, interoperability

The emerging technology of blockchains and distributed ledgers opens the door to an entirely new way to conceptualize interoperability and virtually all aspects of toll system architecture.

This presentation will describe a peer-to-peer system for interoperability that would allow payments to be sent directly from a customer to a toll agency without going through a centralized hub. The benefits and simplicity of an in-person, cash toll transaction can now be realized with electronic toll collection in a distributed network that allows a customer to transact directly with any toll agency, anywhere.

The inherent structure of the model provides a cryptographically secure peer-to-peer network for recording transactions and tracking digital assets and identities in shared, replicated ledgers, such that both the toll agency and the customer are mutually protected from double-billing, double-spending, fraud, and external malicious attacks on the system. The unique design eliminates the need for agencies to share transponder and license plate validation lists and exchange and reconcile interoperable transactions and funds.

This concept not only eliminates the agency-to-agency interaction we experience today, it has has the potential to dramatically reduce the cost of back office operations. Moreover, it can unify tolling, transit, and parking into one customer experience. This presentation will explore blockchain technology, how it is being developed in other industries, its possible uses in toll collection, and the challenges that come with its benefits.
Blockchain and Toll Collection: A Unified Tolling Network

Matthew Milligan
Milligan Partners LLC, Dallas, TX, USA
matt@milliganpartners.com

1 INTRODUCTION

A truly peer-to-peer system for interoperability in electronic toll collection would allow payments to be sent directly from a customer to a toll agency without going through a central party. The benefits and simplicity of an in-person, cash toll transaction can be realized in a distributed network that allows a customer to transact directly with any toll agency, anywhere in the world.

To overcome the challenges of toll interoperability, whether locally or nationally, blockchain technology now provides a solution to unify tolling agencies and transportation user services in general. The inherent structure of the model provides a cryptographically secure peer-to-peer network for recording transactions and tracking digital assets and identities in shared, replicated ledgers, such that both the toll agency and the customer are mutually protected from double-billing, double-spending, fraud, and external malicious attacks on the system. The unique design of the blockchain eliminates the need for agencies to a) share transponder and license plate validation lists and b) exchange and reconcile interoperable transactions and funds.

A blockchain tolling solution would provide for three distributed sets of information that would be shared by the participants in the system: 1) a transaction ledger, 2) an asset inventory, and 3) a list of certified identities. Toll agencies certify suppliers who list their assets—transponders, license plates, and other vehicle identifiers—in the shared inventory. Customers use a digital wallet to store payment instruments, hold prepaid balances, and list their certified assets. When a customer uses a toll facility, the toll agency initiates a transaction directly with the customer’s electronic wallet. The customer participates in the transaction by approving and thus cryptographically signing the transaction.

2 BACKGROUND

The concept of interoperability in toll collection essentially means that one customer, with one transponder account, can seamlessly use the toll facilities of many agencies. Well-known US examples include E-ZPass, Fastrak, and SunPass. This topic comes with some challenges—challenges that industry groups have been working to overcome or minimize since the transponder was first introduced in the late 1980’s. Today, old issues still exist and new ones are being created as operations evolve.

3 THE PROBLEM WITH CENTRALIZATION

Can one organization or company be trusted to act as a middleman for every toll agency in a country or a region? The hub and spoke model can suffer globally from one participant’s mistakes. Those mistakes don’t always make the news, but they exist and the toll industry struggles with solutions to these problems.

Operationally, every toll agency has its own requirements and legislative directives. The different characteristics of toll facilities—roads, bridges, tunnels, and urban, rural, commuter—complicate the landscape. Plus, the cost of doing business, the “the cost to collect,” varies widely in the industry. Every agency calculates transaction processing costs differently. These things combine to be a significant source of difficulty in negotiating interagency agreements. Centralized consortiums require a single set of rules to operate, and a national consortium of all toll agencies will be even harder to wrangle than the current regional collectives. Interoperability needs to support the vast uniqueness within the industry and make participation easy for toll agencies of all shapes and sizes.
4 BEYOND DECENTRALIZATION: A DISTRIBUTED SOLUTION

What if we had another model altogether? Not just decentralized, but truly distributed. What if we could have a distributed system that would allow agencies to use their own rules and cover their unique costs to collect? What if we had a system that could scale to any number of customers and agencies and could eliminate the need for the difficult hub and spoke architecture? A blockchain solution and distributed applications opens the door to these ideas.

Blockchain technology is being developed beyond its initial uses in cryptocurrency. Many of the major financial institutions in the world are exploring the potential uses of blockchains. Efforts extend into a wide array of business models that require transactional accuracy, chains-of-custody, unsurpassed security, and complete transparency. The built-in benefits of blockchain can improve toll collection operations in ground-breaking ways by simplifying processes and drastically reducing operating costs.

5 BACK TO BASICS

Let’s deconstruct a bit. Toll collection began with cash transactions at the toll booth. With cash, the customer physically hands money to the toll collector for the transaction.

Both sides participate and, more importantly, they agree on the terms of the transaction. The customer transacts directly with the toll agency. This type of transaction is ideal for everyone involved, except for one problem—it’s slow... very slow.

Automatic coin machines sped up the process, but the invention of the transponder made things happen rather fast. It was a great leap forward in the world of toll collection. However, the transponder also introduced a great deal of operational complexity. It created the need for a customer account, and it took away the active participation and agreement that happens in a cash transaction.
Toll violations notwithstanding, the transponder is essentially the root of operational complexity and many customer service issues in toll collection. The transponder also presents some of the largest hurdles for interoperability. We need to get back to the simplicity of the cash transaction, but how do we achieve the speed and efficiency of a transponder transaction with the accuracy and trust of the cash transaction… along with all the interoperability solutions we’re looking for?

6 THE NEXT GENERATION OF TOLL COLLECTION

Toll collection started with cash collection, became automated with coin machines, and went digital and wireless with transponders. Now, a fourth generation of operations is possible and it’s founded in the technology of cryptography and blockchains. The blockchain is technology that has the potential to redefine transactions and the back office of a multitude of different industries. In a nutshell, a blockchain is a record of events that is virtually impossible to change. You can record anything in a blockchain, and it becomes an ironclad record of everything that happens to it.

Blockchain technology provides three things that will revolutionize toll collection:
- A shared, replicated, and transparent ledger for all toll transactions
- A secure, unified register of customers, license plates, and transponders
- A method for any customer to transact directly with any agency

This puts the customer back in the center of the picture. By using a shared blockchain architecture, every participant in the system has direct access to the data they need. The need for agency-to-agency data transfers and agency-to-agency financial exchanges for interoperable transactions is eliminated. Every customer can transact directly with every agency.

7 IMPLEMENTING BLOCKCHAIN TOLLING
A blockchain tolling system is made of up a few basic ideas: user certification, asset registration, a transaction ledger, and an electronic customer wallet. The participants in the system are toll agencies, asset suppliers, and customers.

**User Certification**

All participants register through a certification process and are given unique, private digital keys. Agencies and suppliers register and attach to the network by acting as nodes. Customers register and attach to the network through an electronic wallet.

**Asset Registration**

Suppliers and agencies add their assets to the system as they are introduced into circulation. For example, a transponder supplier would add sets of inventories to the blockchain as they are sold to customers. Similarly, as suppliers, Departments of Motor Vehicles would add license plate and registered owner information to the blockchain. (The current state of DMV data presents some hefty challenges, but we believe there are solutions to be found in blockchain capabilities.) Customers use an electronic wallet to store payment instruments, transponder IDs, license plates, and any other potential asset information. The architecture allows for anonymous customers through the private key mechanism, so personal data such as address, phone number, etc. are not necessarily required.

**Transactions**

Toll transactions occur in the lane as they do today. The agency reads a transponder or takes a picture of a license plate. Instead of taking the payment from a customer’s prepaid transponder account, the agency connects directly with the customer’s electronic wallet and initiates a transaction. The customer then approves the transaction and the payment is processed. Transaction approval could be addressed in various ways: customers could manually approve transactions with a button-push; auto-approve transactions from their local agency; auto-approve transactions based on GPS location from a mobile wallet; and auto-approve transactions for certain days and times based on a commute schedule. There are many possibilities for smart and efficient solutions to create a win-win situation for the agency and the customer.

All agencies have access to the entire set of assets and wallets through the blockchain. Agencies would have a real-time connection to what is known, in industry jargon, as the transponder and license plate validation lists, but with even more information at their fingertips. The blockchain contains the complete chain-of-custody for all assets. The blockchain also provides access to a customer’s wallet, which contains the status of payment instruments.

Figure 5 outlines the main principles of operations across participants and the blockchain system.
Figure 5
8 CONCLUSIONS

This simple model describes the use of blockchain for tolling interoperability, and similar models apply to other user fee-based systems like parking, transit, and road user charging. These ideas can be leveraged to improve transportation operations from micro to macro levels, from local solutions to national and international solutions.

Blockchains are at the heart of what drives Mobility as a Service and Smart Cities—open data. Blockchains create new ways to efficiently and safely handle digital identity, digital assets, and data sharing. These models allow detailed trip data to be put into open and anonymous sources of information for public and private entities to use in the betterment of our rural and urban environments. The applications are only limited by our imaginations.
DEVELOPING 21st CENTURY PARKING & TOLLING SOLUTIONS: RE-INTRODUCTION OF HIGHWAY TOLLGATES AND PARKING POLICY IN NIGERIA

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<td>Chief Civil Engineer</td>
<td>Federal Ministry of Power Works and Housing – Works Sector</td>
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<td>Principal Civil Engineer</td>
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E-MAIL (for correspondence) musasaidu38@yahoo.com; niyounge@yahoo.co.uk

KEYWORDS: TOLLING

ABSTRACT:
The development of a good transportation system in World’s developing countries like Nigeria is a major stimulant to the economic growth and development of its citizenry. It ensures the efficient movement of persons and goods in comfort.

This topic is essentially premised on the choice and interest of providing the policy makers in the transportation field the needed and vital information on the opportunities and challenges that may surface if highway tollgates and parking policy are re-introduced in Nigeria. In the research work, there is critical examination of the yardstick behind the establishment of tollgate and parking policy at the initial stage in Nigeria. There was also a reason for its demolition some years back which was for the avoidance of double-taxation. The rationale behind the re-introduction of tollgates and parking policy on the highways was also visited, which rests on the aim of sourcing funds for the maintenance/rehabilitation of highways and for ensuring parking discipline/culture. Basically speaking, various theoretical justifications were used in determining whether toll gates should be reintroduced, in which this research work suggested the adoption of Electronic Toll Collection (ETC) technology so as to guide against time wasting, theft of revenue and traffic congestion. Three toll roads entry/exit ticket systems were also proposed which include; an open toll system, a closed system and open road system.

Recommendations were also made so as to proffer effective enforcement and planning strategies before re-introduction of tollgates and parking policy in Nigeria.
INTRODUCTION

Transport is critical in development of any Nation and the provision of a good transportation network is a key factor in the socio-economic development of a Country. The Nigerian transport system comprises all modes: roads, railways, water, air and pipeline transport. In recent time, Cities in Nigeria have been experiencing a huge surge in population increase mainly due to rising urbanization and rural migration. As a result, these modern Cities face huge challenges in terms of provision of infrastructure as well as coping with the increase in demand for transportation. As the world’s population moves to urban centers, the result is greater traffic congestion, frustrated travelers and lessened productivity. Therefore, an intelligent and actionable transportation system is the key to ensuring that everything runs smoothly and efficiently as possible within the road networks.

In Nigeria, the roads sector (including road bridges) is the dominant mode for intra-state and interstate passenger and goods movement. The importance of the roads sector to the Nigerian economy is apparent from the amount of government capital outlay, the volume of passenger traffic carried and the tonnes of goods evacuated to and from the Nigerian ports. Nigeria has a total road network of 193,200km of which the Federal Government owns 17 percent of the network while the States and Local Government Councils own 16 and 68 percent respectively. As at 2007, only 35 per cent of the Federal roads were rated as being in a good or very good condition. The most recent visual and qualitative condition assessment of Federal roads by Federal Road Maintenance Agency (FERMA) (March 2011) revealed that only 26.5% of the Federal roads are rated as being in a good condition.

Federal roads in Nigeria are deteriorating due to inadequate funding of planned maintenance. The high cost of providing the transport network has traditionally placed the obligation on Government to finance the investment. In an effort to create an efficient and effective transport system, the Federal Government of Nigeria (FGN) is seeking new funding methods to attract needed investment. Additional sources of funding must therefore be identified to fill this gap and tolling system as well as private sector involvement through Public Private Partnership (PPP) are some of the options being considered to rectify the situation.

PURPOSE OF THE RESEARCH WORK

The aims and objectives of this research work is to:
(i) Examine the basis for the initial establishment of tolling and parking systems in Nigeria.
(ii) Examine the reasons for its abolishment
(iii) Establish the rationale behind the need for re-introduction.
(iv) To determine if the re-introduction of toll gates will help provide the needed fund required for the maintenance of the roads.

TOLLING IN NIGERIA

A toll road is a public or private roadway for which a fee (or toll) is assessed for passage (Bhadmus 2015). It is a form of road pricing typically implemented to help recoup the cost of road construction and maintenance. The prominence of toll roads increased with the rise of automobile, and many modern toll ways charge fees for motor vehicles exclusively. The amount of the toll usually varies by vehicle type, weight, or number of axles, with freight trucks often charged higher rates than cars. The widely known and used name for toll collection points in Nigeria is “TOLL GATE”. Its main purpose is for revenue generation which will aid effective and efficient maintenance of federal highways. Apart from the main importance of toll gate which is basically anchored on the generation of revenue for the maintenance and rehabilitation of federal highways in Nigeria, there are so many other benefits attached to which has to do with the spontaneous security they provide to both commuters and transporters because the toll gates were usually managed by armed security agents, their presence provided a level of safety for road users.

Road tolling has become a globally accepted method of raising funds and maintaining road infrastructure at Federal and State Government levels (Rick & Guilmino 2015). It has been tried and tested in many Countries and provides significant economic benefits if properly managed. Over 90 per cent of economic and social activities are run on the Nigerian roads and this has put so much pressure on them. The road transport account for over 90% of the movement of goods and services in the country. The toll systems were first introduced to help generate income for maintaining these roads.

Nigeria had operated some toll roads for several years, but they were abolished in 2004 because of legal issues, revenue leakages and non-maintenance of the tolled roads. In spite of the benefits and successes achieved from road tolling around the world, the one in Nigeria ended abruptly with many unattended consequences. The first tolling system was introduced the so called oil-boom era and this quickly turned to a nightmare due to gross mismanagement.

On the economic side, the toll gates soon became drain pipes. Instead of generating revenue for the maintenance of the road as anticipated, it rapidly became a source of fund for corrupt officials handling the tolls. At the peak of it all, some of the toll attendants even made more money than the Government who hired them by selling their own tickets as against the one provided by the Government. Besides, some Government agencies such as the Police and Customs use the toll gates for stop and search operations where motorists are interrogated, thereby causing untold hardship, disruption of journey and often times extortion.
The last tolling regime in Nigeria before it was scrapped attracted many ugly sights. The toll gates were turned into a trading point where different goods and services were sold. Even beggars and touts had a filled day doing their business. This activities led to a huge chunk of garbage with unpleasant smell and the degradation of the environment. More worrisome were the activities of armed robbers who capitalize on the reduced speed of vehicles at the toll gates to rob unsuspecting motorists. The toll gates then became hide outs for criminals and armed bandits (Omorotionmwan, 2016).

Nigerians cannot forget so quickly also the traffic jams experienced at the toll gates during festive seasons where queue of vehicles stretches for a long distance. In some cases, motorists sleep on the road inside the car as a result of the logjams. A journey of a few hours soon extend to days making motorists spend the yuletide on the road. These are some of the challenges that characterized the previous tolling regime and it is important to look at this pitfalls and other ones before the re-introduction of new toll gates

WAY FORWARD

Having looked at the challenges that characterized the previous tolling regime, it is imperative to look at ways of addressing the pitfalls in order to achieve a successful tolling system in Nigeria. Since the idea of re-introducing the toll gates on the Nation’s highways, there have been mixed reactions to the development. Some Nigerians support the idea while others are still skeptical owing to the massive fraud and mismanagement associated with the previous tolling regime. It is important this time around to look for ways to curb corruption and loop holes where accumulated revenues from the toll gates are looted. Also, Government agencies must be prevented from using the toll gates as a base for extorting and manhandling innocent road users.

In addition, a committee should be set up to engage with all stakeholders both (primary and secondary) and ensure to come up with blue print on the best possible ways for the realization of an efficient and effective toll gating system. With the tolling system once again being considered as a policy option in Nigeria, the Country has the advantage to assess successfully tolling models in order to develop its own tolling framework which will help effectively manage and maintain our road networks.

RATIONALE FOR TOLLING

The basis for the re-introduction of tolling on Nigerian roads was born out of the need to generate revenue to fix the Nation’s highways which have long become very deplorable. Due to poor allocation of funds over the years to the Federal Ministry of Works which is responsible for the construction and maintenance roads in the country, about 80% of the 36,000km federal roads in the country are severely damaged having exceeded their life span. Most of the roads now require total reconstruction and rehabilitation. Many lives have been lost in Nigerian roads due to their bad conditions and this has also significantly affected economic activities since the roads are the main mode of transporting people, goods and services.

The proposition of the toll gates is seen as an alternative source of generating revenue to maintain the federal roads which has been affected due to inadequate funds. Government spending has been hampered as a result of dwindling oil prices which has been the Nation’s main source of finance, leading to economic recession. Hence, the need to find alternative sources of funds to augment Government spending on roads which is grossly inadequate. Tolling is becoming the 21st century solution of choice for generating additional user-based transportation revenue (Olu, 2011). This proven source of revenue is being seriously considered for expanded use by the Federal Government, Cities and even States with support from elected officials across the political spectrum.

STAKEHOLDER ENGAGEMENT

For tolls to achieve its purpose, there must be proper engagement with all stakeholders both within and outside the transportation sector. Since the cost of financing these investments will be met by road users through tolls and other commercial revenue as appropriate, it is imperative to have a clear policy for tolling and processes for its implementation. Extensive public consultation with all stakeholders must be conducted to ensure public acceptance. This is necessary to forestall a similar incident that happened in Lagos, Nigeria on the Lekki Expressway which was built through Public Private Partnership (PPP) and later became a problem when it was opened to public with motorist protesting about alleged tolling of the road. Fortunately, the incident was brought under control and the road has since been tolled so as to recoup the returns on investment.

Private investors must be confident that the appropriate policies are in place and the legitimacy of the toll collection and enforcement is ascertained. Presently, there is a Green paper themed: Federal roads and bridges tolling policy in Nigeria, which sets out the framework for effective re-introduction of tolling in Nigeria. The Green paper which was formulated between October and November, 2013 is a consultative document which allows a wider range of stakeholders both in the public and private sector to provide their input in the formulation of the tolling policy. All inputs are to be carefully considered and synergized to produce the proposed policy. In response to the views expressed and concerns raised, the Green paper will be re-developed into a draft white paper which is submitted for legislative backing.
LEGAL FRAMEWORK

In Nigeria, only the national assembly has the power to re-introduce toll gates as an additional source of funding road maintenance. The Federal Government has no direct power to re-introduce toll gates on Nigerian toll gates unless the national assembly gives legislative approval in parliament (Ocheyenor, 2015). The Executive Government must also seek legislative approval for the design of a compatible payment platform to cover the Country. Nigerian laws permit some form road and bridge tolling but legislation must be improved to provide a modern framework for management of toll roads which covers both public sector tolling and those built through Public Private Partnership (PPP).

Presently, there is a legislation currently before the National Assembly to establish the Federal Road Authority which will be responsible for promoting a safe and efficient management of federal road network with a view to meeting the socio-economic demands of the Country. Also, there is a bill to establish a National Road Fund which shall be a repository for revenues accruing from road user charging systems and other sources for the purpose of financing the maintenance and upkeep of national roads and promote sustainable development of the road sector.

The proposed fund would provide predictable and sustainable management of road networks. A governing board shall be established for the management of the fund and create an enabling environment for private sector participation, management and financing in the road sector. This bills if passed into law will go a long way in enhancing road sector development in Nigeria and provide a sustainable framework for the maintenance of roads and bridges across the Country.

ROAD TOLLING CONCEPTS

Road tolls were levied traditionally for a specific operation (e.g. city) or for a specific infrastructure (e.g. roads, bridges) repair and maintenance (Gilliet, 1990). These ideas were widely used until last century. The evolution in technology made it possible to implement road tolling policies based on different ideas. The different charging ideas are designed to suit different requirements regarding purpose of charging, charging policy, the network to the charge, tariff class differentiation, etc. On this note there are three types of Road Tolling ideas which are explained below:

1. Distance or Area Charging: This is one of the types of road tolling concepts. In a distance or area charging system concept, vehicles are charged per total distance driven in a defined area.

2. Time Based Charges and Access Fees: In a time-based charging system concept, a road user has to pay for a given period of time in which he may use the associated infrastructures. For the practically identical access fees, the user pays for the access to a restricted zone for a period of several days.

3. Motorway and Other Infrastructure Tolling: This type of tolling concept is used for charging well-defined special and comparatively costly infrastructures, like a bridge, a tunnel, a mountain pass, a motorway concession or the whole motorway networks of a country. Classically a toll is due when a vehicle passes a tolling station (toll gate), be it a manual barrier-controlled toll plaza or a free flow multi-lane station. (Adewunmi, 2008)

TOLLING ENTRY/EXIT SYSTEMS

There are two systems of tolling entry/exit commonly used in Nigeria which include: Open (with mainline barrier toll plazas); Closed (with entry/exit tolls). Modern toll roads often use a combination of the two systems, with various entry/exit tolls supplemented by occasional mainline tolls. These systems of tolling entry/exit aforementioned are explained below:

1. Open system: Here, all vehicles stop at various locations along the highway to pay a toll. While this may save money from the lack of need to construct toll booths at every exit, it can cause traffic congestion while traffic queues at the mainline toll plaza (toll barriers). It is also possible for motorists to enter an ‘open toll road’ after one toll barrier and exit before the next one, thus travelling on the toll road toll-free. Most open toll roads have ramp tolls or partial access junctions to prevent this practice known as “Shunpiking”.

2. Closed system: Vehicles collect a ticket when entering the highway. In some cases, the ticket displays the toll to be paid on exit. Upon exit, the drivers must pay the amount listed for the given exit. Should the ticket be lost, a driver must typically pay maximum amount possible for travel on that highway. Short toll roads with no intermediate entries or exits may have only one toll plaza at one end, with motorists traveling in either direction paying a flat fee either when they enter or when they exit the toll road. In a variant of closed toll system, mainline barriers are present at the two endpoints of the toll road, and each interchange has a ramp toll that is paid upon exit or entry. In this case, a motorist pays a flat fee at the ramp toll and another flat fee at the end of the toll road; no ticket is necessary. In addition, with most systems, motorists may only pay tolls with cash and/or change; debit and credit cards are not accepted. However, some toll roads may have travel plaza with Automated Teller Machines (ATMs), so that motorists can stop and withdraw cash for the tolls. The toll is calculated by the distance travelled on the toll road.
Recently, the Federal Government took the tolling system into account and indicated that this approach might not be the best option for the country. The last tolling regime in Nigeria was through public sector funding, and lessons learned from the collapse of this system underlines the importance of carefully selecting a viable road for tolling. Tolling of roads can either be done solely through public sector funding or private sector funding, or a combination of both, which is the public-private partnership. Toll roads are very sensitive to financial strains; traffic volume may fall considerably. This is attributed to the funding of road infrastructure delivery. Funding road infrastructure projects remains a major constraint in the delivery of efficient and improved road networks across the country. The finance of road projects has been through the traditional annual budgetary allocation which has proved to be grossly inadequate. Information from

**ELECTRONIC TOLLING COLLECTION (ETC)**

This aims to eliminate the delay on toll roads by collecting tolls electronically. The main functions of Electronic Tolling Collection (ETC) are listed below:

1. Determines whether the cars passing are enrolled in the program.
2. Alerts enforcers for those that are not.
3. Electronically debits the accounts of registered car owners without requiring them to stop.

ETC systems rely on four (4) major components:

1. **Automated Vehicle Identification (AVI):** It is the process of determining the identity of a vehicle subject to tolls. The majority of toll facilities record the passage of vehicles through a limited number of toll gates. At such facilities, the task is then to identify the vehicle in the gate area.
2. **Automated Vehicle Classification (AVC):** This is closely related to AVI. Most toll facilities charge different rates for different types of vehicles, making it necessary to distinguish the vehicles passing through the toll facility.
3. **Transaction processing:** Deals with maintaining customer accounts, posting toll transactions and customer payments to the accounts, and handling customer inquiries. The transaction processing component of some systems is referred to as a “customer service center”. In many respects, the transaction processing function resembles banking, and several toll agencies have contracted out transaction processing to a bank.
4. **Violation enforcement:** A violation enforcement system (VES) is useful in reducing unpaid tolls, as an unmanned toll gate otherwise represents a tempting target for toll evasion. Several methods can be used to deter toll violation:
   - Police patrols at toll gates
   - Physical barriers, such as gate arm
   - Automatic number plate recognition.

ETC has facilitated the concession to the private sector of the construction and operation of urban freeways. Also, it has made feasible the improvement of road congestion pricing schemes in a limited number of urban areas to restrict auto travel in the most congested areas (Copeland, 2008).

In 1959, Nobel Economics Prize winner, (Vickrey, 1992) was the first to propose a system of electronic tolling for Washington Metropolitan Area. He proposed that each car would be equipped with a transponder: The transponder’s personalized signal would be picked up when the car passed through an intersection, and then relayed to a central computer which would calculate the charge according to the intersection and the time of day and add it to the car’s bill.

Enforcement is accomplished by a combination of a camera which takes a picture of the car and radio frequency keyed computer which searches for a driver’s window/bumper mounted transponder to verify and collect payment. The system sends a notice and fine to cars that pass through without having an active account or paying a toll.

The most revolutionary application of ETC is in the urban context of congested cities, allowing to charge tolls without vehicles having to slow down. This application made feasible to concessions to the private sector the construction and operation of urban freeways, as well as the introduction or improvement of congestion pricing, as a policy to restrict auto travel in downtown areas.

**TOLLING FINANCE/RETURN ON INVESTMENT**

The application of public-private-partnership together with the use of tolling provides Nigeria with an excellent strategy for maintaining its roads for the benefit of the Nigerian economy, businesses and Communities. Federal roads are deteriorating due to inadequate funding of programmed maintenance. Additional sources must therefore be identified to fill this gap and private sector investment through PPP is one source which should be considered to rectify the situation. Notwithstanding the relatively poor condition of Federal roads in Nigeria, traffic volume continue to increase. Nigeria’s vehicle population has increased rapidly over the years. This is an indication that demand for road services will continue to grow especially in urban areas (Federal Ministry of Works, 2013).

Traffic volumes are sensitive to income and economic growth. At this time of recession in Nigeria and considering that toll roads are very sensitive to financial strains, traffic volume may fall considerably. This underlines the importance of carefully selecting a viable road for tolling. Tolling of roads can either be done solely through public sector funding or private sector funding, or a combination of both which is the public-private partnership. The last tolling regime in Nigeria was public sector funding and lessons learnt from the collapse of the tolling system indicate that this approach might not be the best option for the country.

Recently, the Federal Government took bold steps in ensuring the rehabilitation, maintenance, re-construction and expansion of major arterial highways in the country. Against this backdrop, there have been numerous challenges confronting Government effort in delivering improved road infrastructure. One of the major challenges as stated earlier is attributed to the funding of road infrastructure delivery. Funding road infrastructure projects remains a major constraint in the delivery of efficient and improved road networks across the country. The finance of road projects has been through the traditional annual budgetary allocation which has proved to be grossly inadequate. Information from
the Federal Ministry of Works indicates the annual funding requirements for roads is estimated as NGN500bn (Nigerian Naira) over the next ten years against an average budgetary allocation of NGN120bn, giving a shortfall of NGN380bn. These shortfall have proven to have dire negative consequences on road development in the Country. As a result it is imperative for the Government to reduce dependence on public finance by facilitating private sector participation in road development. Hence, the need for the Federal Government of Nigeria to turn to private investors to share the risks, costs, financing, constructing and operating road infrastructure.

By assessing private investment enabled or repaid tolling system, much of the federal road network can be improved more rapidly instead of relying on the traditional sources of financing. Provided the programme is well managed, federal toll roads can reduce journey times, travel costs and ensure safe travel for road users. PPP or Public-Private Partnership which is funded and operated through a partnership of government and one or more private sector companies (Obozuwa 2011) is considered to be the best approach towards tackling the current and impending challenges facing the development of the road infrastructure in the Nigeria. With the establishment of the Infrastructure Concession Regulatory Commission (ICRC) through the ICRC Act of 2005, the federal Government of Nigeria has demonstrated its commitment towards this objective. The ICRC is the agency of the Federal Government of Nigeria responsible for catalyzing public private partnerships for the development and implementation of a world class PPP framework towards the development of a world class infrastructure projects for the benefit of Nigerians and the Nigerian Economy.

Most developed Countries such as the United Kingdom, China, United States of America, Germany and France have adopted PPP model not only to develop and grow their economies but also as a way of fixing the infrastructural gap. There are enormous benefits to be achieved by Nigeria if they can follow suit on this PPP framework as well. It must be said that some PPP model have been applied in the maritime and aviation sector in Nigeria, which to some extent have successful but more needs to be done in the areas of policy formulation and legislation to ensure that they are sustainable. Some key benefits of PPP to both public and private sector investment are:

Key benefits of the PPP for private investors in Nigeria are:

- It enables access to long term investment opportunities with the backing of the Federal Government of Nigeria.
- It gives opportunities for private sector partners to expand their businesses on successful track record has been established.
- Private sector partners will also have the opportunity to achieve efficiency based on financial, technical and innovative capability.

Key benefits to the public sector include:

- Economic stability is maintained when private investors fund infrastructural development without the Government taking loans and paying back with interests.
- The public sector can gain from the expertise and innovation of the private investors.
- Higher growth rate is achieved through additional investment and increased production capacity from the participation of private sector.

ANALYSIS OF A PARTICULAR ROAD PROJECT IN NIGERIA.

In order to embark on the concession of major highway roads through the Public Private Partnership arrangements, it is important to conduct feasibility studies on technical, legal, social, environmental and financial viability of the road projects for government partnership with the private sector investments. The result of this assessment are compiled to produce an outline business case which is used by private investors to make business decisions. This report does not seek to produce a comprehensive business case but rather provide an insight using analytical representation and brief discussions to determine the financial viability of a selected road in Nigeria.

The methodology adopted in generating this report and analysis is based in site investigation, socio-economic profile analysis, traffic surveys and forecasts which is then used to make informed decision about investing in the road. In line with the objective of this report, this section aims to showcase the potential investment opportunities in road infrastructure projects in Nigeria. The road selected for study and analysis is the Abuja-Lokoja road in North Central geo-political region of Nigeria.

PROJECT TITLE:               ABUJA – LOKOJA ROAD PROJECT

ROAD TYPE:                                  203km Four –lane dual carriageway separated concrete New Jersey barrier
GENERAL SPECIFICATION: Flexible pavement with asphaltic concrete or rigid pavement;  
Lane width of 3.65 minimum  
Carriageway width of 7.3m  
Shoulder width of 2.75m  
Design Speed of 100 km/hr

PROJECT RATIONALE:

The proposal for improvement of the existing 4-lane dual carriageway includes provision of the following components; Geometric Improvements; Widening Proposal; Service Roads; Side walk: Longitudinal Profile improvement; Improvement of Junctions; Bridge and Cross Drainage Structures; Special Problems and; Traffic Control and Safety Measures

SOCIO-ECONOMIC PROFILE ANALYSIS

The Abuja – Lokoja road alignment is considered to be an important highway and lifeline for trade and commerce. The 203km road is an ongoing dualized carriageway with lay byes in designated areas along the stretch of the road. The road is a well-travelled highway that connects the Southern parts of the country to the Northern part including Abuja- the Federal Capital city. Apart from the political relevance of the Abuja – Lokoja route, it further provides a link for transportation of agricultural produce from the Southern part of Nigeria to the Northern region and vice versa. Therefore, any effort aimed at improvement of the route will catalyze growth and development of economic activities across the region.

TRAFFIC SURVEY ANALYSIS AND FORECASTS

An investigation of the traffic volume along these routes were conducted in order to establish the traffic flow characteristics, travel pattern, users’ willingness to pay toll, economic and financial viability, junction improvements, road safety components etc. The projected traffic volume on the Abuja – Lokoja road alignment plays a critical role in the private public partnership transaction as it determines to a large extent the technical and financial viability of the proposed investment. The traffic survey of the Abuja – Kaduna - Kano route was carried out using the Automatic Traffic Count (ATC).

AUTOMATIC CLASSIFIED TRAFFIC COUNT SURVEY

Automatic Classified Traffic Count Survey was carried out with the use of Metrocount Vehicle Classification System device-MC5600. The MC5600 is a simple axle-based pneumatic counter/classifier which collects data at a 'Time-Stamping'. The principle of operation of the device is such that every axle hit by the vehicle moving at least 10km/hr is being recorded at the time of hit by a pneumatic sensor. The recorded data is then interpreted by specialized computer software to give an output. These studies were conducted at two selected regular count stations for twenty-four hours over a 7 day period along the length of the Abuja – Lokoja route alignment.

The table below represents the traffic volume data at two different count stations on each section of the route based on information on the annual average daily traffic (AADT) for different classes of vehicles.

| MetroCount Traffic Executive
| Daily Classes
| Datasets: |
| **Site:** [ABUJAJ-LOKOJA SOUTH BOUND PT1] |
| **Direction:** South bound A>B, North bound B>A. |
| **Number of Lanes:** 2 |
| **Survey Duration:** 6:25 Thursday, October 13, 2016 => 6:59 Saturday, October 22, 2016 |
| **Identifier:** CP14VEQR MC56-L5 [MC55] (c) Microcom 19Oct04 |
| **Data type:** Axle sensors - Paired (Class/Speed/Count) |
| **Profile:** |
| **Included classes:** 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12 |
| **Speed range:** 10 - 160 km/h. |
| **Separation:** All - (Headway) |
| **Scheme:** Vehicle classification (ARX) |
| **Units:** Metric (meter, kilometer, m/s, km/h, kg, tonne) |
| **In profile:** Vehicles = 104796 / 105198 (99.62%) |
Table 1. Traffic Volume Data at count station 1

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**Average daily volume**

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| **Weekdays** |     |      |      |      |      |      |      |      |      |      |      |      |       |
| (%)   |      |      |      |      |      |      |      |      |      |      |      |      |       |

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| **Weekend** |     |      |      |      |      |      |      |      |      |      |      |      |       |
| (%)   |      |      |      |      |      |      |      |      |      |      |      |      |       |

|       |      |      |      |      |      |      |      |      |      |      |      |      |       |
| **Average daily volume** |     |      |      |      |      |      |      |      |      |      |      |      |       |
| **Entire week** |     |      |      |      |      |      |      |      |      |      |      |      |       |
| (%)   |      |      |      |      |      |      |      |      |      |      |      |      |       |

|       |      |      |      |      |      |      |      |      |      |      |      |      |       |
| **Weekdays** |     |      |      |      |      |      |      |      |      |      |      |      |       |
| (%)   |      |      |      |      |      |      |      |      |      |      |      |      |       |

|       |      |      |      |      |      |      |      |      |      |      |      |      |       |
| **Daily Classes** |     |      |      |      |      |      |      |      |      |      |      |      |       |

<p>| <strong>Datasets:</strong> | [LOKOJA-ABUJA PT1 NORTH BOUND] |
| <strong>Direction:</strong> | North bound A&gt;B, South bound B&gt;A. |
| <strong>Number of Lanes:</strong> | 2 |
| <strong>Survey Duration:</strong> | 6:42 Thursday, October 13, 2016 =&gt; 7:17 Saturday, October 22, 2016 |
| <strong>Identifier:</strong> | CN95K584 MC56-L5 [MC55] (c) Microcom 19Oct04 |
| <strong>Data type:</strong> | Axle sensors - Paired (Class/Speed/Count) |
| <strong>Profile:</strong> | Included classes: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12 |
| <strong>Speed range:</strong> | 10 - 160 km/h. |
| <strong>Separation:</strong> | All - (Headway) |
| <strong>Scheme:</strong> | Vehicle classification (ARX) |
| <strong>Units:</strong> | Metric (meter, kilometer, m/s, km/h, kg, tonne) |
| <strong>In profile:</strong> | Vehicles = 109154 / 109625 (99.57%) |</p>
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Average daily volume

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Average daily volume

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<td>(%)</td>
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<tr>
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<td>(%)</td>
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<td>81.2</td>
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</tbody>
</table>

From Table 1 & 2 above, it can be observed that the Average Daily Traffic (ADT) volume on both sides of the carriageway exceed 10,000 vehicles per day. Experience in Nigeria has shown that not all roads are good and viable for tolling. Hence, the need for a clear objective criteria in selecting roads for tolling. One objective criterion is the average daily traffic (ADT) carried by a road. As shown in Table 3 below; indicative daily traffic requirement for tolling, it is generally accepted that the required traffic for a road to be financially viable may range from 1,500 to 15,000 vehicles per day depending on the cost to be recovered. (Draft Green Paper, 2013)
Table 3. Indicative Daily Traffic Requirements for Tolling

<table>
<thead>
<tr>
<th>Daily Traffic Requirement</th>
<th>Costs to be recovered</th>
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<td>15,000 – 25,000</td>
<td>New construction / reconstruction</td>
</tr>
<tr>
<td>6,500</td>
<td>Rehabilitation</td>
</tr>
<tr>
<td>3,500</td>
<td>Maintenance</td>
</tr>
<tr>
<td>1,500</td>
<td>Recovery of toll collection costs</td>
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The implication of the traffic survey report from the Abuja-Lokoja road indicates there exists tremendous investment opportunity if tolling is introduced on the road given the high traffic flow of passengers and freight movement. Both sections of the road project is considered to be economically viable considering they have an average daily traffic of between 9,000 to 12,000 vehicles per day.

CONCLUSION

The provision of roads and transportation facilities as well as their maintenance are fundamentally important. It contributes to the development of a Country as well as the well-being of its inhabitants. Nigerian roads need urgent attention and one of the main sources of generating revenue for such maintenance is by re-introducing toll gate facilities in Nigeria. It has been established that funding road infrastructure through the annual budgetary allocation has proven to be unsustainable. More so, the economic implication of the decay of existing road infrastructure has had a negative impact on the Citizenry. Consequently, the Federal government of Nigeria need to embark on securing alternative mechanism of financing road infrastructure development other than the usual meager annual budgetary allocation.

Introduction of tolling is a way of meeting the critical road requirements of the country. The tolls can best be established through a public private partnership and funds invested recouped through tolling of the roads. At the same time, the toll income is also used to manage and maintain the road and provide ancillary services. There are substantial benefits to be derived from tolling, provided certain principles are adhered to. Nigeria has long suffered from a sub-optimal road network that imposes significant costs in terms of travel time, vehicle wear and tear and high accident rates. By introducing the new and modern electronic tolling collection system, much of the Federal road network can be improved and expanded more rapidly and more extensively than would be possible using traditional sources of financing.

Now that tolling is once again being considered as a policy option, Nigeria has the advantage to usefully refer to the experience and good practices of other countries, learn from its earlier mistakes during the last tolling regime and apply these lessons in successfully building, managing and maintaining its toll roads. It is evident from the findings of this research work that the re-introduction of federal toll gates in Nigeria is an essential tool for the generation of vast amount of income which can be used for the rehabilitation and maintenance roads. Tolling of roads is a global phenomenon which all must embrace provided it translates to good and safer roads in our country.

RECOMMENDATION

Consequent upon the findings of this research, it is therefore recommended that;

- Government must on its part ensure that the toll gates are operated in a transparent manner so as to avoid the problems associated with the operation of the toll gates in the past.
- Toll concessions should be entered only where such concessions are financially-viable.
- Mandatory public consultation should be introduced prior to tolling of any road or bridge in Nigeria to provide users with critical information in advance of tolling being introduced.
- Weighbridges should be introduced at toll gates to prevent overloading of vehicles plying the highway which is the main cause of road damage.
- The indices of corruption and lack of accountability must be properly taken care of, to avoid the situation where the Police and Customs Officers use the toll gates as bases for the extortion of innocent motorists.
- Alternative route should be provided for motorists who are unwilling or unable to pay for toll.
- Further research relating to this study should be embarked upon, in order to facilitate a successful tollgate’s management system.
REFERENCES


Olu, F. (2011). “Returning of Tollgates: This government is wasteful, Nigerians Say No!” Vanguard Newspaper: www.vanguardngr.com


Introduction

The constraints of the national budget were the spur for the introduction of tolls in the early 1980’s on portions of the declared national road network in South Africa. The road network in South Africa comprises of some 747,000 km allocated between the three spheres of government – national, provincial and local authority (municipal). Of this total, the declared national road network consists of 22,000 km, of which 3,132 km is tolled, all of them being inter-city / provincial routes in non-urban environments. The tolls on these roads were collected manually (cash and credit cards) by means of conventional manually-operated toll plazas.

At the country’s economic heart is Gauteng province – a mere 17,100 sq. km in area (1.4% of South Africa) but responsible for generating approximately 35% of South Africa’s gross domestic product (GDP) and 10% of Africa’s GDP, Gauteng is therefore one of the most important economic areas and ensuring its sustainability is paramount, enabled in part by an efficient, productive transportation system.

The volume of the traffic on the busiest highways within Gauteng required a toll collection system that would allow vehicles to travel unimpeded and the tolls collected
Conventional toll plazas would wreak havoc on any thoroughfare – in South Africa or elsewhere - carrying between 100,000 and 230,000 vehicles per day. It was quite clear that imposing tolls to fund much-needed capacity expansions and ongoing maintenance could not depend on conventional approaches. Employing toll plazas was not an option.

Approval of the implementation of the Gauteng Freeway Improvement Project (GFIP) comprising 201 km of improvements to the declared national route in an urban environment provided an opportune moment to adopt new operating principles. Although this was primarily a road infrastructure upgrade programme that was aimed at providing additional road capacity to accommodate growing economic demands, the volume of traffic dictated that a different operating model from the then existing one had to be adopted. The ZAR 22bn (approx. USD 2bn) civil works project, comprising 12 individual works contracts awarded over a 3 year period was funded through raising capital on the capital markets – by issuing bonds – to ease congestion, reduce journey times, improve journey time reliability and provide the motorist with a safe driving experience. The improvements were agreed by government to be funded by tolls, in line with the established ‘user-pays’ principle.

**Options**

At the outset, it was planned that on this portion of the declared national road network the Multi-Lane Free Flow (MLFF) method, also known as open road tolling (ORT), would be adopted as an operating principle.

MLFF enables tolls to be paid electronically at highway speeds simply by driving under a gantry using either an e-tag fitted on the inside of the vehicle’s windscreen or ‘video tolling’ based on the Vehicle Licence Number (VLN) as an account identifier. This type of Electronic Toll Collection (ETC) already used in other countries ensures high levels of operational efficiency without imposing any delays on the high volumes of traffic. To ensure high levels of compliance and as a deterrent to non-payment, the enforcement strategy is based on capturing evidential-quality images to prove that a vehicle was present on the road network, setting an escalating tariff regime and prosecuting non-payment offences through the local courts.

At the outset, it was also decided that the GFIP programme would include a platform for national interoperability i.e. the use of one e-tag on the entire toll road network.
throughout South Africa, irrespective of who the toll operator is. This also ensures a consistent quality of service for everyone using the tolled routes - and it also allows new and existing toll road operators to benefit by not having to invest in back office solutions nor the administrative and legal infrastructure associated with this.

Constituent Components

There are three components to the national tolling scheme: A Road Side System (RSS) deployed on road segments subject to tolls, a Transaction Clearing House (TCH) and a Violation Processing Centre (VPC). The TCH enables efficiency savings through economies of scale with the incorporation of the existing toll road network, future expansions within the country and as an enabler for regional ETC schemes. The VPC delivers a standardized approach to debt collection and enforcement. The RSS records each transaction on the road as it takes place and transmits the information to the centrally located TCH. Figure 1 shows the constituent components for the interoperable system.

Figure 1: Constituent Components
An interoperable ORT system required a comprehensive but pragmatic approach to its roll-out. It had to be robust to withstand any legal challenges and be user-friendly both for the operator and the road user.

In this context, the objectives of the development of the national ETC regime were to:

- define ETC standards for South Africa;
- conduct an interoperability study and establish the necessary standards;
- design the TCH, RSS and VPC components and related operating strategies;
- draw-up the functional specifications;
- embark on an educational campaign on the use and benefits of the system; and
- define and execute a transparent and internationally competitive procurement process.

The design of the system was such that it would instill a level of confidence in road users that the benefits of tolling are greater than the tolls themselves. For example, enforcement is positioned towards debt collection rather than the imposition of punitive penalties and the related marketing efforts were directed towards gaining an insight into the different categories of road users, raising awareness of GFIP, developing an understanding and securing the users’ acceptance.

As the implementing authority, the South African National Roads Agency Limited (SANRAL) had no doubt that the process would be multi-layered and complex, mitigated by the fact that the technical and operating principles for ORT had already been established elsewhere and that there was sufficient local and international expertise to ensure a successful implementation – and overall that GFIP was deliverable.

**Standardization**

In order to ensure the implementation of a robust and reliable system, standards had to be set that would allow multiple vendors to participate with equal opportunity in order to elicit comparable bids. This entailed an initial study process to determine the international best practice with respect to the technologies for charging and enforcement, the most appropriate approach to contract packaging and to ensure seamless interfacing with other national service providers.
Stakeholders, users, suppliers and regulators were invited to submit proposals on the standards to be employed for ETC in South Africa. The choice of a universal standard was to ensure that all electronic tolling was uniform throughout the country and interoperable across various toll operators including at conventional toll plazas where ETC had already been rolled out. The elected public domain standard is based on the Dedicated-Short Range Communication (DSRC) specifications developed by the Comité Européen de Normalisation (CEN), 5.8GHz; that was ultimately branded as e-tag.

The CEN standard was adapted for South African conditions. This standard was found to be the most reliable interoperability. Since this standard has been in use in Europe, Australia and South America for a number of years. It has proven itself to be reliable and there were numerous suppliers that could supply the system at competitive prices at the time of initial procurement and thereafter. The selection of the standards not only ensures transparency in the procurement process but also derisks it since additional or replacement tags confirming to the standards that define e-tags could be procured from other suppliers, a benefit that we exercised during the initial procurement and thereafter.

Importantly, a standard was specified and not a specific charging technology employed by a specific system or offered by a specific technology provider. The universal standard levels the playing field among technology suppliers that enables any technology supplier whose products comply to provide the required equipment such as tag readers, tags, and cameras.

**Framework**

A framework for electronic toll collection was established that allowed for structured outcomes of the consultations between the roads authority and various stakeholders. The outcome was a set of agreed policies for:

- tolling operations;
- technical interoperability;
- data transfer specifications;
- security and privacy specifications; and
- proforma agreements between concessionaires and the roads authority.
In order to accommodate the then existing ETC system at the conventional toll plazas elsewhere in South Africa, it was agreed to set up a steering committee amongst the roads authorities and the concessionaires. The purpose of the steering committee was to ensure the implementation of an interoperable ETC system throughout South Africa with a single tag being accepted by all concessionaires. The function of the steering committee was to:

- facilitate the process to achieve interoperability;
- determine baseline principles and common business rules;
- review framework documents;
- reach consensus and accept framework documents and principles; and
- commit members to sign an interoperability agreement.

The work of the steering committee was assisted by work groups who reported to the steering committee where agreements were verified and signed.

The works groups were:

- legal;
- commercial;
- organizational
- transaction clearing and financial;
- marketing;
- distribution;
- technical; and
- enforcement.

The deliberations culminated in the acceptance of the following principles:

- no further implementation of non-interoperable ETC tolling systems;
- TCH operated as a not-for-profit entity operating on business principles with full cost recovery;
- central transaction processing;
central account management and tag procurement;
central database with distributed access;
pre-paid and guaranteed post-paid accounts;
image-based enforcement and universal provision of services, permitting concessionaires to opt in or out of these;
TCH and VPC operated on least cost, optimal return basis;
centrally managed pre-paid float;
maximize account adoption and ETC penetration;
high level courteous customer service;
common technology standards and operating specifications;
predictable levels of system performance and business continuity provisions;
certification process to ensure minimum standards and compliance;
compliance with applicable security, privacy standards and policies;
auditable system; and
compliance and governance in accordance with statutes.

As above, the objectives of providing a highly standardized central office i.e. the TCH and VPC, was to reduce the barriers to entry for toll road operators wishing to offer ETC and to enable interoperability throughout the country.

Transaction Clearing House

The TCH can be regarded as the ‘engine’ room of the system. Therefore, it is important that the TCH provides a consistent and accurate service; is able to process an account to which multiple tags are registered, and accept the unique tag of each vehicle at all conventional plazas as well. As the TCH is the sole source of transactional data it must be able to accurately and efficiently settle payments received through the National Payment System (NPS) linking the all commercial all banks. Furthermore, the TCH must provide accurate information to the customer through the Internet at all times. The information on the customer and any e-tags registered to customer account must also be accessible to the customer service centre.

The TCH is also responsible for clearing all electronic transactions between the respective toll operators and the customer. The TCH is also responsible for the
management of all registered accounts, the call centres, maintaining a consistent quality of service provided by customer service outlets and managing the logistics associated with the tags – from ensuring sufficient inventory and delivery of tags to retailers, individuals and all other toll road operators – and any tags returned for whatever reason.

A logical, progressive sequence is followed to record and reconcile tolling transactions and related toll liabilities at the TCH from start to finish with account holders and with road users identified by means of their VLN s. The back office at the TCH receives the information on the passage of the vehicle under the gantry (See Figure 2) from the Road Side System (RSS), where the e-tag was been read, together with an image of the front, back and top of the vehicle (for evidentiary purpose when required) in support of the volumetric classification regime; the details are verified as per the registered account and toll fees allocated to the relevant account without the vehicle stopping or slowing down. At the point where the tag and its associated vehicle is first detected and identified, it is not known if the vehicle is associated with a registered e-tag or if there are sufficient funds in the related account. This function is carried out at the clearing stage of the account in the TCH.

Figure 2: Tolling Gantry
Two other actions take place at this back office. Discounts and exemptions on the toll tariffs are applied to qualifying accounts and related vehicles and manual number plate recognition is carried out if the Automatic Number Plate Recognition (ANPR) (See Figure 3) fails. The failure could be due to several reasons, including their being no number plate attached to the vehicle, the number plate being obscured or it not being possible to transact with a registered e-tag on the vehicle. On completion of these tasks, including exception conditions, the transaction, including any related images and their meta data, is forwarded for processing at the TCH. The TCH also directly receives all the transactions from the other toll road operators and concessionaires complying with published business rules, including requirements on content and timeliness of such transactions.

Figure 3: Manual Validation
The transactions received at the TCH are checked against the relevant accounts to see if there are sufficient funds in such accounts. If the funds in the account are insufficient, or the vehicle does not have a registered account or, the vehicle had no number plate the matter is forwarded to the VPC for further processing.

Considering the broad array of functions carried out at the TCH, it can easily be appreciated that, a central TCH that provides the above services including a central financial clearing arrangement results in lower costs to toll operators, concessionaires and any future parties that join the ‘club’ as opposed to the plethora of TCHs that would need to be established by individual operators and / or concessionaires with their concomitant impact on road users. Lowering the costs of operations ensures the sustainability of the national tolling regime whilst allowing for continual improvements in customer quality of service and accessibility of payment and enquiry channels by all users of tolled routes.

**Violation Processing Centre**

The VPC is responsible for debt recovery from recalcitrant road users. The VPC is charged with ensuring that any infringement notices that are sent out are accurate and sent to the correct defaulting party. Prior to sending on an infringement notice personnel at the VPC have to verify the following:

- manually confirm the existence and accuracy of the interpretation of the vehicle’s number plate from images collected against a data base maintained elsewhere;
- obtain (if vehicle not linked to an account at the TCH) or check vehicle owner’s details – this too from a database residing elsewhere;
- compile, print and mail invoice (where the electronic address of the defaulter is not known);
- monitor debt collection process and report on its progress; and
- send out reminders.
The VPC is also responsible for handling representations made by the motorist, judge the merits of the representation made and decide whether to accept or reject the representation made.

If the latter, it elects to proceed with litigation and informs the vehicle owner accordingly. In this instance, the VPC is responsible for the preparation of the evidentiary material required to proceed to a law court to the satisfaction of the court. Thus, the VPC is the interface between the toll operator and the law enforcement authorities.

Conclusion

ETC provides substantial advantages over manual toll collection, including savings on operations and external environmental costs. The use of DSRC technology for ETC is proven and is priced competitively on an end-to-end basis and on whole life costs due to the highly competitive nature of the market for technology, systems integration and operations.

The RSS and back office operations are underpinned by a highly automated workflow that is based on accurately recording the passage of each vehicle and manual checks of evidential-quality images in the event that the electronically recorded vehicle information is inaccurate or incomplete. Vehicles are classified by volume and shape, paying a differentiated tariff.

The TCH is responsible for issuing tags and accounts, clearing transactions and customer relations management. The VPC handles the debt collection, enforcement operations and supports external debt recovery processes. The TCH and VPC are both located at a central location linked to every conventional toll plaza and gantry in the entire country.

The TCH and VPC provide a centralized national service and thereby enables national interoperability, currently serving an ETC network of 3120 km, of which 545 km are MLFF, operated by a multitude of operators and concessionaires. The underlying ETC technology is based on the 5.8 GHz specifications developed by TC278, a technical group within CEN. This tolling regime is one of the world’s first nationally scaleable multi-operator, interoperable systems in the world underpinned by public domain standards, represents a landmark in improving road operations, delivers superior
customer quality of service and reduces the entry costs for new toll schemes anywhere in South Africa.

### Abbreviations

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
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<tbody>
<tr>
<td>ANPR</td>
<td>Automatic Number Plate Recognition</td>
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<tr>
<td>CEN</td>
<td>Comité Européen de Normalisation</td>
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<tr>
<td>DSRC</td>
<td>Dedicated-Short Range Communication</td>
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<td>ETC</td>
<td>Electronic Toll Collection</td>
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<td>GDP</td>
<td>Gross Domestic Product</td>
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<td>GFIP</td>
<td>Gauteng Freeway Improvement Project</td>
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<tr>
<td>GHz</td>
<td>Giga Hertz</td>
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<tr>
<td>ITS</td>
<td>Intelligent Transport System</td>
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<td>MLFF</td>
<td>Multi-Lane Free Flow</td>
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<td>NMC</td>
<td>Network Management Centre</td>
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<td>NPS</td>
<td>National Payment System</td>
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<tr>
<td>ORT</td>
<td>Open Road Tolling</td>
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<tr>
<td>RSS</td>
<td>Road Side System</td>
</tr>
<tr>
<td>SANRAL</td>
<td>South African National Roads Agency (Limited)</td>
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<tr>
<td>TCH</td>
<td>Transaction Clearing House</td>
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<tr>
<td>VLN</td>
<td>Vehicle Licence Number</td>
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<tr>
<td>VPC</td>
<td>Violation Processing Centre</td>
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</table>
Acknowledgements

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An Analysis of Road Safety Legislation across the Globe

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Abstract

Legislation of key risk factors of road safety along with adequate enforcement has a significant importance in current discussion on traffic safety. This study carried out an in depth analysis of the level of advancement of safety legislation across the world. A secondary descriptive analysis of the 2013 Global status report on road safety is attempted to assess legislation on five risk factors (drinking and driving, helmet use, seatbelt use, child restraints, and speeding) in all six WHO regions. The descriptive analysis was performed for 170 countries with laws classified as per the following benchmark: 1) existence of legislation and 2) the level of enforcement of legislation based on a subjective threshold (0-5 for low enforcement, 6-7 for medium enforcement and 8-10 for high enforcement, all on 0-10 Scale). The socio economic characteristics for each region were taken on board by stratification of regions on basis of income groups. Study results revealed that European region is observed to have at least 90% of the countries having legislation for all five key risk factors. Also, it is revealed that helmet usage appears to be the most extensively enforced legislation with 32.35% of countries having high enforcement level globally. Child restraints appear to be the least enforced with only 11.17% of countries having high enforcement level of child restrain law.
1. **Introduction**

Injuries, fatalities and deaths resulting from traffic accidents have increased the inimical impact of human life retardation with each degree of increase in motorization. The World Health Organization reports that 1.24 million people were killed globally due to road traffic crashes (WHO 2009). Recent developments in ever-increasing number of road traffic injuries (RTIs) results in RTIs as eight leading cause of death worldwide. Therefore, legislating five of the main risk factors (usage of seatbelts and child restraints, the wearing of helmets for two wheeled motorized vehicles, driving against alcohol influence, and most importantly speeding) together with adequate enforcement is essential in improving road safety situation. A considerable amount of literature has concluded increasing speeds, non-use of helmets, drink-driving and failure to implement safety laws as special factors responsible for RTIs (Mohan 2002, Suriyawongpaisal and Kanchanasut 2003, Gururaj 2004, Dandona et al. 2005, Gururaj 2005). This expanding alertness was addressed by several studies that reported the exigency of adequate enforcement critical to mandating road safety key risk factors (WHO, 2004; PAHO, 2004; WHO, 2009; Hijar, 2012). For example, while assessing five key risk factors for road safety, *World Report on Prevention of Road Traffic Injuries (RTIs) in 2004* by the *World Health Organization (WHO)* emphasized on enforcement of road safety legislation (WHO, 2004). In the same year while investigating road safety legislation in 20 countries of Americas region, Pan American Health Organization concluded well organized and structured legislation as effective component of balanced road safety policy. Also, the prevalence of different laws at national, sub-national, and municipal level (inconsistency in road safety laws) was highlighted and suspected for counter-productivity respectively (PAHO, 2004). However, the assessment of level of progression(s) of road safety legislation worldwide was difficult before a more comprehensive and rigorous approach was adopted by WHO (WHO, 2009). In addition, one of the other complication was the inability to assess the depth to which the implementation of extensive approach recommended by WHO was being achieved by countries. Following up the recommendations through conducting a global survey of 178 countries, a standardized and
detailed dataset was compiled by WHO that contained information about existence, adequacy, and progression of road safety legislation respectively (WHO, 2009). While several researchers in past performed various statistical and analytical studies to evaluate the road safety legislation, the prominent work done for analyzing all five key risk factors was performed by (Híjar et al. 2012). A secondary analysis was performed for Americas region by utilizing data from 2009 Global Status Report on Road Safety: time for action (WHO, 2009). The study concluded seat-belt law as the most extended legislation and speeding law to be the least extended legislation respectively. The study also affirmed the need for adequate laws and the affirmation of law be based on what is known to be more effective. Despite the valuable and interested findings, the study focused on performing descriptive statistics for only 30 countries of the Americas region. In addition, no research has explicitly focused on analyzing the global road safety legislation respectively. Thus, this paper seeks to address the afore-mentioned research gap by formulating structured study with an aim to focus on the comparison and level of advancements of key risk factors across the six WHO regions. A statistical analysis of five risk factors (drinking and driving, helmet use, seatbelt use, child restraints, and speeding) is carried out.

2. Material and Methods

2.1. Study Population

In order to assess the global road safety legislation, data file has been extracted from 2013 Global Status Report on Road Safety (GRSS): Supporting a Decade of Action. This specific report builds on the world report on RTI prevention by WHO (2004) and WHO report (2009) in an attempt to evaluate the progress of participating countries in implementing the recommendations of 2009 world report (WHO, 2013). WHO (2013) performed the extensive survey in 182 participating countries for distinguishing the inconsistencies in road safety programs and further to identify potential interventions. However, twelve countries are eliminated from the process of descriptive analysis due to missing and incomplete data about the key risk factors. Through this study, 93.40% of the world’s population in six different regions (Africa, America, South East Asia, Europe, Eastern Mediterranean Region and Western Pacific Region) has been analyzed for legislation and enforcement on five key risk factors respectively.

2.2. Data collection and Input Variables
A structured interview questionnaire was sent to all participating countries where National Data Coordinators (NDCs) steered and managed the data collection process. Individual respondents from different public and private road sector and health organizations in each country further facilitated the data collection process. National Data Coordinators were assigned the task to nominate up to eight professional experts for completion of the questionnaire. The structured interview questionnaires provided opportunity to local experts for remarking on availability of key risk factors along with their subjective perception of the enforcement level(s) of each key risk factor. For example, the experts independently rated the enforcement of key risk factors on a scale 0-10 with zero and ten corresponding to not effective and highly effective respectively. For each participating country, the individual perceptions of respondents (on Scale 0-10) regarding enforcement level(s) of key risk factors were amalgamated in NDCs group meetings to come up with concrete set of information that best represented the country’s effort for legislation of key risk factors. It is noteworthy that WHO followed specific criteria for devising and evaluating presence of comprehensive laws for five key risk factors. For instance, comprehensive laws for speeding, drink driving, helmet use, seat-belt usage and child restraints were defined as: existence of national level speed limit law with an urban speed limit of either 50 km/h or less; existence of drink driving law with blood alcohol concentration (BAC) being 0.05 g/ dL or less; existence of helmet law that covers all motorized two- or three- wheel riders; existence of seatbelt law and its application to both front- and rear-seat occupants; and presence of child restraint law respectively.

2.3. Methods

A descriptive analysis is performed in this study, calculation of frequencies and percentages for the categorical and continuous variables extracted from WHO 2013 using econometric software LIMDEP (INSERT REFERECE HERE). For this study, the descriptive analysis is performed with laws classified on a scale as per the following benchmark: 1) existence of corresponding law for a specific key risk factor and 2) the level of progression of each law based on a subjective threshold (0-5 on scale 0-10 for low enforcement, 6-7 for medium enforcement and 8-10 for highly enforced legislation respectively). The scale value of ≥ 8 for high enforcement of legislation is fixed in accordance with WHO guidelines (WHO, 2013). The socio economic characteristics for each country are also considered for stratification of countries and regions on
the basis of income levels. As per definition of WHO, 33 countries are low income countries with a GNI per capita of less than 1045$, 94 countries conform to middle income countries with a GNI per capita of more than 1045$ but less than 12746$, while a total of 43 countries correspond to high income economies with a GNI per capita of 12746$ or greater. Six regions: Africa, America, South East Asia, Europe, Eastern Mediterranean Region and Western Pacific Region were studied for the existence and enforcement levels of five key risk factors. Figure 1 illustrates the distribution of low, middle, and high income countries in six regions globally.

![Figure 1 WHO regional groups on basis of income, 2013](image)

3. Results

A total of 12 (out of 39) African countries have legislation for the five risk factors. Approximately, 68% (21 countries) and 90% (45 countries) exhibited legislation for five key risk factors in Americas and European regions respectively. Approximately 46% of countries in Western Pacific Region possessed key risk factors legislation respectively. Exactly one country, each in South East Asia and Eastern Mediterranean Region, exhibited legislation for all key risk factors simultaneously. Table 1 and Table 2 summarizes the presence and/or absence of legislation and enforcement level for five key risk factors.

3.1. Speeding
Table 1 indicates that a total of 36 African countries have national legislation on speeding, and three countries (Congo, Comoros and Tanzania) have legislation set at sub national level. Low level of effectiveness of enforcement level of speed limits is observed in 87% (34 countries) of the African countries. Out of these countries, 54% and 33% of countries correspond to low and middle income groups respectively.

A total of 25 countries in Americas region have their speeding legislation set at national level. Similarly five countries in Americas region have legislation set at sub national level with Dominica, Dominican Republic and Venezuela conforming to middle income while United States of America and Canada conforming to high income countries. Furthermore, 65%, 26% and 9% of the countries in Americas region have low, medium and high level of speed limit enforcement respectively.

All the ten countries in the Southern Eastern Region have legislation for speed limits set at national level. Nine countries have low level of speed limit enforcement out of which 33.33% and 66.66% are low income and middle income countries respectively. One middle income country, Maldives, has a medium level of speed limit enforcement in South East Asian region.

The analysis indicates that all fifty countries in European region possess legislation for speeding at national level. European countries have relatively better enforcement levels with 14%, 60% and 26% of countries possessing low, medium and high level of enforcement respectively. Of the 30 countries possessing medium level of enforcement, 36.66% correspond to middle income countries while 63.33% correspond to high income countries.

Table 1 indicates that 18 countries of the Eastern Mediterranean region (EMR) own legislation for speeding at national level. A total of eight EMR countries possess low level of speed limit enforcement, while seven countries have medium level of speed limit enforcement. Jordan and Syrian Arab Republic possess middle income economies while United Arab Emirates possesses high income status, all of which reported enforcement to be equal to or greater than 8 on a scale of 0-10 (high level of enforcement).

This study also investigated the existence of speeding legislation for 22 countries in the Western Pacific region, out of which 91% of the countries have legislation set at national level while Australia and Micronesia have speeding legislation set at sub national level. Lao, Cambodia and Solomon Islands are the low income countries in this region that exhibit a speeding legislation at national level. Table 2 indicates the level of enforcements of speed limit
law in the Western Pacific region with 40.9%, 45.45% and 13.6% (Australia, New Zealand and South Korea) of the countries having low, medium and high level of speeding enforcement respectively.

3.2. Drink Driving

Approximately 95% of the participating African countries have legislation on drinking and driving (DD) while 5.12% (Sao Tome & Principe and Togo) of the countries lack such legislation for drinking and driving. Additionally, 92.30% of the African countries possess low level of drink driving enforcement. Out of all the 36 countries having low level of DD enforcement, 63.88% and 36.11% of the countries correspond to low and middle income groups. However, Swaziland and Botswana rated their enforcement as 6 and 7 on scale 0-10 respectively, which is high given the average (3.20 on scale 0-10) enforcement level in African region.

Table 1 Existence of legislation for key risk factors

<table>
<thead>
<tr>
<th>Regions</th>
<th>ROAD SAFETY KEY RISK FACTORS</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>Drinking and driving</td>
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<tr>
<td>WHO region</td>
<td>Income level</td>
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<tr>
<td>All</td>
<td>All</td>
</tr>
<tr>
<td>Low</td>
<td>All</td>
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<tr>
<td>Middle</td>
<td>All</td>
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<td>High</td>
<td>All</td>
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<tr>
<td>African Region</td>
<td>All</td>
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<tr>
<td>Low</td>
<td>All</td>
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<tr>
<td>Middle</td>
<td>All</td>
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<tr>
<td>High</td>
<td>All</td>
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</table>
The totality of 31 countries in the Americas region has legislation on drinking and driving with legislation set at sub national level for USA and Paraguay. With no low income countries in this region, 59.25% and 50% of the middle and high income countries have low level of enforcement for this key risk factor, thus identifying the need for initiating enforcement programs focusing on dangerous proportion of drivers.

Out of the ten countries in the Southern Eastern Region, nine have legislation for drink driving set at national level. Maldives is a country with no legislation on drink driving. Low level of enforcement is found in all nine countries, out of which 33.33% and 66.66% are low and middle income countries respectively.

<table>
<thead>
<tr>
<th>Region</th>
<th>All</th>
<th>Low</th>
<th>Middle</th>
<th>High</th>
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<tr>
<td><strong>South East Asia</strong></td>
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<tr>
<td><strong>All</strong></td>
<td></td>
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<td></td>
</tr>
<tr>
<td><strong>Low</strong></td>
<td>9</td>
<td>0</td>
<td>1</td>
<td>10</td>
</tr>
<tr>
<td><strong>Middle</strong></td>
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<td>1</td>
<td>2</td>
<td>0</td>
</tr>
<tr>
<td><strong>High</strong></td>
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<td>1</td>
<td>27</td>
<td>0</td>
</tr>
<tr>
<td><strong>South East Asia</strong></td>
<td>9</td>
<td>1</td>
<td>27</td>
<td>0</td>
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<tr>
<td><strong>Europe</strong></td>
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<tr>
<td><strong>All</strong></td>
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<td>0</td>
<td>50</td>
<td>0</td>
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<tr>
<td><strong>Low</strong></td>
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<td>2</td>
<td>0</td>
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<tr>
<td><strong>Middle</strong></td>
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<td>21</td>
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<tr>
<td><strong>High</strong></td>
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<td>0</td>
<td>27</td>
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<td><strong>Eastern Mediterranean Region</strong></td>
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<tr>
<td><strong>All</strong></td>
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<td>0</td>
<td>16</td>
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<td><strong>Low</strong></td>
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<td>10</td>
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<td>6</td>
<td>0</td>
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<tr>
<td><strong>Western Pacific Region</strong></td>
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<tr>
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<td>0</td>
</tr>
<tr>
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<td>1</td>
<td>11</td>
<td>0</td>
</tr>
<tr>
<td><strong>High</strong></td>
<td>5</td>
<td>1</td>
<td>5</td>
<td>0</td>
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</tbody>
</table>

Note: S-N: existence of law at sub-national level.
All the low, middle and high income countries in European region have a legislation made for drink driving at national level. European countries have relatively better enforcement levels with 20%, 36% and 44% of countries possessing low, medium and high level of enforcement accordingly. Surprisingly, Tajikistan is a low income country in this region with a sufficiently higher enforcement of 9 on a scale 0-10. More specifically, out of all road traffic crashes, only 2 percent of the crashes recorded involve drunk drivers (WHO, 2013).

A total of 17 (94.44%) countries in EMR have legislation for drink driving law while Afghanistan is the only country with no legislation so far. Similarly, 44.44%, 33.3%, and 22.22% of the countries in this region possess low, medium and high level of drink driving enforcement respectively.

All the 22 countries in the Western Pacific region have legislation for drunk driving, with Australia and Micronesia having legislation set at sub national level. Three countries (Fiji, Tonga and Vanuatu) and two countries (New Zealand and South Korea) are the middle and high income countries respectively with a medium level of enforcement for this risk factor. This highlights the heterogeneous effect of socio-economic status on corresponding effectiveness of enforcement programs.

3.3. Helmet

Out of 39 countries in the African region, 36 countries have legislation on helmet law at national level. Burundi, Gambia and Kenya lack such legislation for helmets usage. This explicitly adds to the larger proportion of vulnerable road users getting injured as a result of RTC (WHO, 2013). Also, keeping in view the severe injuries associated with bicyclists and motorcyclists, preventive interventions such as helmet usage and reflective jackets could potentially reduce the severities of injuries respectively (Eric, 2011). Additionally, 66.66% of the African countries possess low level of helmet enforcement while 17.94% and 15.38% of the countries have medium and high level of helmet enforcement respectively.

On the other hand, 96.77% of the countries in Americas region have legislation for helmet law with Canada, Mexico and USA having helmet legislation at sub national level. Dominica is a country with no helmet legislation at all. Low, medium and high level of enforcement is present in 38.70%, 32.25% and 29.03% of the countries respectively.
All the ten countries in Southern Eastern region possess legislation for this risk factor. Moreover 40% of the countries in this region have medium level of enforcement. Bhutan, Indonesia and Maldives are the countries with high level of helmet enforcement. Another study also concluded better compliance of motorcycle drivers (almost 80-91% drivers using helmets) in Indonesia as compared to all Southeast Asian nations (Peltzer, 2014). Furthermore, three middle income countries (Bangladesh, India and Timor-Leste) possess low level of enforcement.

All the low, middle and high income countries in the European region possess legislation for helmet law. A relatively higher proportion of countries (48%) possess high level of helmet enforcement. Low and Medium level of enforcement is found in 20% and 32% of European countries respectively.

The analysis revealed that 88% of the countries in EMR possess legislation on helmet usage. Afghanistan and Yemen are the low and middle income countries in this region respectively with no legislation for this risk factor. A total of ten countries possess low level of helmet enforcement, out of which Kuwait and Saudi Arabia (high income groups) have low level of helmet enforcement consequently. Also, low income working males expatriates are more likely to be vulnerable victims in RTFs in Arab countries (Al-Shammari et al 2009). Alarmingly, approximately all pedestrian involved fatalities included head injuries which further leads to wonder one if all vulnerable road users should be wearing helmets for necessary safety (Bahloul et al. 2009, 2004). In addition, 27.77% and 16.66% of the countries have medium level and high level of helmet enforcement respectively.

A good majority (95.45%) of the countries in the WP region hold legislation for helmet usage with Australia and Micronesia having the law at sub national level. Kiribati is a middle income country in this region with no legislation on helmet usage. Eight countries (36.36%) of the WP region possess low level of helmet enforcement. On the other hand, 18.18% and 45.45% of the countries possess medium and high level of enforcement respectively.

3.4. Seatbelt

Seatbelt legislation exists in 76.92% (30 countries) of the African countries with the rest of the countries lacking legislation for seatbelt law. A broad majority of the countries (66.66%) have low level of seatbelt enforcement, with 10.25% and 23.07% of the countries having medium and high level of seat belt enforcement.
Intuitively, all the middle and high income countries in the Americas region possess legislation for seatbelt law. Low level of enforcement is found in 13 countries (all middle income), with 35.48% and 22.58% of the countries having medium and high level of enforcement respectively.

Nine countries of the Southern Eastern region have legislation on seat belt law. Myanmar is the only country in this region with no legislation for this key risk factor. Majority of the countries (7 countries) in this region exhibit low level of seat belt enforcement. West Bank & Gaza strip and Maldives possess medium level of enforcement while Indonesia is the only country with high level of seat belt enforcement respectively.

Seatbelt legislation at national level is observed for all the countries (all income groups) in the European region. Furthermore, 18%, 42% and 40% of the countries have low, medium and high level of seat belt enforcement respectively in this region. Interestingly, Kyrgyzstan is the only low income country possessing high level of seat belt enforcement. On the contrary, three countries (Belgium, Denmark, and Slovakia) are the high income countries having low level of seat belt enforcement respectively. Therefore, more efforts (intensive enforcement actions, highly visible and well publicized) need to be concentrated for improving driver’s compliance to seat belt usage (ETSC, 2011).

Seat belt legislation is found in 94.4% of the countries in the Eastern Mediterranean region. Afghanistan is the only country with no seat belt legislation at national level. Similarly, 44.4%, 16.66% and 38.88% of the countries in this region possess low, medium and high level of enforcement respectively. Bahrain and Kuwait are the high income countries with low level of enforcement respectively.

Specific to Western Pacific region, 81.18% of the population has been covered with seatbelt legislation. However, four countries (Micronesia, Palau, Solomon Islands and Tonga) lack such legislation. Moreover, 54.54% of the countries possess low level of enforcement for this risk factor, with medium level of enforcement in 22.72% of the countries. High level of enforcement is found in New Zealand, Philippines, Samoa, Singapore and South Korea respectively.

3.5. Child Restraints
More than 66.66% of the countries in the African region reported lack of legislation for obligating the use of child restraint devices for children in vehicles. Only 33.33% of countries possess legislation for child restraints. All the 39 countries possess low level of child restraint enforcement respectively.

Approximately 64.74% of the countries in Americas have legislation for this key risk factor with Canada, Mexico and USA possessing law at sub national level. Furthermore, 80.64% of the countries exhibit low level of enforcement with only 9.67% of countries having medium and high level of enforcement respectively.

Nine out of ten countries in the Southern Eastern region have no legislation for child restraint. Timor-Leste is the only country in this region possessing legislation for this risk factor. With no legislation for child restraint in nine countries, all the countries have a 0 enforcement level (very low) on scale 0-10.

Intuitively, 90% of the countries in European region possess such legislation for child restraint. Low level of enforcement is reported in 40% of the participating countries. While medium and high level of enforcement is reported in 32% and 28% of the countries respectively.

Majority (94.44%) of the countries in Eastern Mediterranean region have no legislation for child restraints with Saudi Arabia is the only country possessing such legislation and that too with a low level of enforcement. Accordingly, all the 18 countries exhibit low level of enforcement respectively.

Legislation on child restraints is reported in exactly half of the countries in the Western Pacific region. Australia is the only country possessing legislation at sub national level. Low and medium level of enforcement is observed in 77.27% and 13.63% respectively. New Zealand and Samoa are the only countries with high level of enforcement respectively.

4. Discussion

This effort complement few other efforts to analyzing legislation on the key risk factors for RTIs across the globe by covering vast majority of member countries of WHO 2013. However, previous work acclimatized the development of this study on road safety (Hijar et al, 2012). Through this study, it was possible to observe different trends of existence of legislation and respective enforcement across various regions.

4.1. Africa
Out of 39 countries, 12 countries possess legislation on all five key risk factors. According to the results of this study, the risk factor having legislation in most of the countries was speeding, with 100% of participating countries having legislation, followed by drinking and driving (94.87%), helmet use (92.30%), seatbelt use (76.92%), and child restraint (33.33%). Child restraint is the least addressed risk factor with only 13 countries having legislation on this risk factor. The risk factor best addressed in terms of high enforcement level is seat belt use, with 23.07% of countries having high enforcement, followed by helmet use (15.38%), speeding (5.12%), and drink driving (2.56%). Child restraint is the least addressed risk factor in participating African countries, as no country reported to have high level of enforcement for this risk factor. An overall valuation suggests that preeminent efforts should be made in the region to enforce existing laws for the five risk factors associated with RTIs. Furthermore, drink driving law and speeding should be accoutered with appropriate enforcement programs and actuated awareness campaigns. The large number of countries with low level of enforcement can be attributed to low income, inadequate political environment and weak institutional framework for improvement of traffic safety. Accordingly, the low income of African countries reinforces the inability of the institutions to initiate and monitor adequate policy for the prescribed problem at hand (Chisholm et al. 2012). The intricacy of RTIs in the region suggests that law enforcement should be accompanied with improvement of safe road infrastructure.

4.2. Americas

According to the results, 67.74% of the countries in this region possess legislation on all five key risk factors. The risk factor best addressed in terms of high enforcement level is helmet use, with 29.03% of countries having high enforcement, followed by seatbelt use (22.58%), drunk driving (12.90%), and speeding and child restraints each in 2.56% of the participating countries. The average Gross National Income per Capita (2010) for this region is 7690$ which is consistent with middle income group. This factor is reflected in relatively better enforcement of laws as compared to that of African region. The average drink driving enforcement level was 5 in the participating countries. The exclusion of pedestrians and motorcyclists in formulation of drinking driving law is a critical factor for the safety of the users as they have a high risk of being engaged in a crash (Desapriya et al. 2003). Thus, more work should be done in future to address the safety of the vulnerable road users (usually low income groups) by incorporating
them in the drink driving legislation. Motorcyclists are appearing in a noticeable amount on the road networks of Colombia and Suriname, which further propound the plausible transformation of RTIs in the region (Hijar et al. 2012). The improvement and augmentation of existing laws is also a critical factor to be practiced together with higher levels of enforcement for ensuring adequate road safety in this region.

4.3. South East Asia

Timor-Leste is the only country in this region with legislations on five key risk factors. Drinking and driving, child restraint and speeding are the risk factors with very low (0 on scale 0-10) level of enforcement in this region. However, 30% of the countries have high level of enforcement for helmet use with another 10% of countries having high level of enforcement for seat belt usage. The overall low income level of the region is likely to add to the inability of the region to maintain adequate amount of law enforcement. According to (WHO 2013), vulnerable road users i.e. motorized two or three wheelers, pedestrians and cyclists account for nearly 50% of the deaths in this region. This key fact highlights the essence of reinforcing the drinking and driving, speeding, and helmet use laws with optimal levels of enforcement. Due to the heavy share of two and three wheelers in the road traffic share in this region, the authorities need to address the threatening situation by policies in place that are implemented to promote non-motorized and public transport and thus to separate vulnerable road users as a way of protecting them (WHO, 2013).

4.4. Europe

Approximately 90% of the countries in this region possess legislation on five key risk factors. With only two countries having low level of income, majority of countries exhibiting medium and high level of enforcement are reported to be middle and high income countries respectively. Only 48% of the countries in this region rate their helmet-use enforcement level as high (8 or higher, on a scale 0-10), followed by drinking and driving (44%), seatbelt use (40%), child restraints (28%) and speeding (26%). These better levels of law enforcement in Europe (EU) can be attributed to better comprehensive laws for five key risk factors, stringent enforcement, relatively high income, and appropriate road safety strategy. Yet there seems potential room for improvement in the enforcement levels of key risk factors. If average driving speeds were to drop by only 1km/hr on all roads across the EU, more than 2200 road deaths...
could have been prevented each year, with 1000 of them on rural roads, 1100 on urban roads and 100 on motorways. Similarly, another 2500 deaths could have been prevented if 99% of the occupants had been wearing a seat belt (ETSC 2011). Thus, the introduction of enforcement plans with yearly targets may further improve driver’s compliance to speed limit, drink driving and seat belt law respectively.

Table 2 Law enforcement for key risk factors

<table>
<thead>
<tr>
<th>Regions</th>
<th>Drinking and driving</th>
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<th>Seatbelt</th>
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4.5. Eastern Mediterranean Region

A relatively larger proportion of countries (61.11% and 33.33%) in this region belong to middle and high income status. 18 out of 22 countries of the Eastern Mediterranean region are studied in this research that corresponds to 81.81% of the Region’s population. High level of seatbelt enforcement is reported in 38.88% of the participating countries, followed by drinking and driving (22.22%), and helmet use and speeding (16.66% each). 17 of the 18 countries have seat belt laws, but Iraq, Morocco, Oman, Saudi Arabia, Sudan, Syrian Arab Republic and UAE are the only countries that report their seat belt enforcement as high. On the other hand, 10 countries in the region prohibit the use of alcohol, yet Iraq, Saudi Arabia, Syrian Arab Republic and UAE are the only countries that report a high level of DD law enforcement respectively. This situation alarms the threat posed to road user’s safety due to poor level of DD law enforcement. Additionally, Syrian Arab Republic, Pakistan, Iran, Egypt and Afghanistan are the countries with noticeable amount of motorized two or three wheeled vehicles on the roads (WHO 2013). Yet the average helmet enforcement in all these countries is 2.8 on a scale 0-10, which enlightens the poor level of helmet legislation and respective enforcement levels, further posing a major threat to the vulnerable road users. 83.33% of the countries in this region have low or medium level of speeding enforcement respectively. The governments in this region thus should ensure a mutually cooperative transport system for enhancement of safety by taking into account all users and all forms of transport (WHO 2013).

4.6. Western Pacific Region

The statistical analysis revealed that 10 of 22 countries in the Western Pacific region possess legislation on all key risk factors. High level of helmet use enforcement is reported by 45.45% of the participating countries. Alarmingly, 65% of the people killed on roads in high income countries are vulnerable road users (riders and passengers of motorcycles, pedestrians and cyclists) (WHO 2013). On the other hand, 31.8% of the countries possessed high level of enforcement for drink driving law. High level of enforcement for seatbelt use, speeding and child restraints was found in 22.72%, 13.63% and 9.09% of the participating countries respectively.
The safe integration of vulnerable road users into multimodal transportation system can be achieved by considering the safety and mobility needs of motorcyclists, cyclists and pedestrians.

**References:**


TITLE:  THERE IS STILL SOMETHING TO LEARN ABOUT DRIVING
TRACK:  Towards Zero Fatalities and Serious Injuries
AUTHOR:  Carlo ROSSI
POSITION:  Founder of DRIVING CAMP and DRIVE AT BEST
ORGANIZATION:  DRIVE AT BEST
COUNTRY:  MONACO
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INTRODUCTION

I am Carlo Rossi. I founded the “Driving Camp” Academy and “Drive at Best” Online Courses. I would like to talk about the main cause of deaths on the roads.

When a plane crashes, causing hundreds of deaths, the news travels around the world. It’s right that it’s like that, because every life is priceless.

But few know that 3500 (!) people die every day in road accidents around the world.

As if one plane crashed every hour!

And we have the sad total of more than 1.2 million deaths per year. Like in a world war!

But we mustn’t forget that a part of the deaths and injuries include pedestrians and cyclists who are run over, probably without any responsibility. The number of deaths worldwide among the so called “vulnerable road users” is 800 per day, for a total of 300.000 a year! … like the entire population of Iceland.
OBJETIVES

The solutions that are proposed, both by governments and by every agency or company involved in traffic management, can be summarized as follows:

Reducing speed limits and increasing checks.

Improving road conditions: wider with better asphalt and larger curves. Roads protected by guardrails, with better signs and lighting.

Technological evolution of the car: stronger deformable structures. Also active and passive safety systems and driving aids. In the not so immediate future, effective and reliable self-driving cars.

Education and improvement of driver conduct and respect of the traffic rules.

They are all valid topics, which have led to interesting results over the years.
But far too few are dealing with driving skills of the drivers. It’s assumed that all of them drive well. **Or maybe we think that they cannot improve?**

We would like to focus on this last point, which is crucial for us. In fact, most accidents are caused by humans. Setting speed limits, improving roads and cars is definitely useful. But as long as the drivers make mistakes, there will always be accidents which can result in injuries and death.

And if we ask drivers to judge their own driving skills, especially car lovers, we find that they are all convinced that they drive well! Is this true? Unfortunately no.

Now let’s reason this out together. For someone who wants to fly planes, there are several different pilot’s licences. From an ultralight to a jet, the training course is very challenging.

There is also more than one driving license for lorries and motorcycles, with different theoretical and practical training courses.

The same thing for boats, and if the boat driver decides to dive in and explore the seabed using a SCUBA tank, here too there are different licenses, each with specific training.
And there is even more than one license to fly a small drone!

So why can someone drive any car from a FIAT 500 to a Ferrari anywhere in the world with the same driver's license?

On top of it, many drivers, having passed the driving test, are convinced that they are driving champions. In reality, they drive more like Mr. Magoo than Hamilton or Vettel. Do you remember him? Like the elderly man in the cartoons, lots of motorists run many risks without realizing it.

Before becoming a Captain, airplane pilots must record numerous hours of flight time and take specific training. They initially learn the basics of flying, then they experience all the potentially dangerous situations using simulators and in real planes.

But going back to the Driver's License: Does this mean that you really know how to drive, or is it just a permit to get around?

And here's an important question: Do Driver Training Schools really teach you how to drive?
They teach the basics of driving, including manoeuvres and parking, and how to drive in traffic. And, of course, they teach drivers to respect the rules, traffic signs and other motorists.

They cannot do any more than that due to time restraints and, above all, cost.

This is because the lessons at driver training schools around the world follow the instructions from the competent government offices.

But did someone who obtained a Drivers Licence really learn how to drive? This may seem like a provocative question, but the answer, in our opinion, is far from obvious. And it’s not yes.

A novice driver has probably learned how to move about on the roads. This driver will have to teach himself the rest by driving.

Learning to read and write as children began by going to school. Anyone who wants to graduate has to study up to 25 years.

Why so long? Clearly, there is a lot to learn!

Getting a driver's license is a bit like finishing elementary school. Like reading and writing, you learn the basics of driving. But there is still so much more to learn!

If in doubt, just think:
If a novice driver perceives a danger, would he/she be able to correctly react to the emergency situation?
Did anyone ever teach the driver that? Of course not!
But fortunately, after a few years of experience driving, everything changes...

Sure about that? What evidence is there that this novice driver learned the manoeuvres that no one ever taught him and has never tried to do?

Then, when accidents occur is called fate...
It is the fate of people who are driving around, without knowing how to drive!

And the result is 1,200,000 deaths per year on the world’s roads.
As we have seen, there are many remedies to improve this tragic situation.

We should not only punish drivers with fines, but give them appropriate training.
Train them not only with practical Safe Driving tests, which would be the best way, but also with online courses, which are more convenient and economical.

CONCLUSIONS
So the goals of our "Driving Camp/Drive at Best” Academy are:
First, try to increase Governments awareness of the importance of investing on advanced training of drivers, as indeed has already been made with planes, lorries, motorcycles, boats, drones...
Second, to teach you to drive well by means of our practical courses (where, unfortunately, only a few thousand students can attend each year). They cost a few hundred Euros, but are more effective than life insurance!...

Third, for those who cannot participate in practical courses, to provide a useful and interesting E-learning tool. The "Drive at Best" On-line Coaching, which a result of more than 30 years’ experience, is convenient, accessible, inexpensive and is available in multiple languages.

The courses are divided into three main topics:
- Defensive/Behaviour
- Eco + Care Driving
- Safe Driving

If I could have a moment of your time, I would like to ask you to take a quiz, one of the many included in our online Courses. It is very informative and impressive at the same time.
Presenting Abu Dhabi’s New Lighting Standards for Improving Driver and Pedestrian Visibility across all Road Categories

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1 INTRODUCTION

2017 brought the implementation of the second and final phase of Abu Dhabi’s Sustainable Lighting Strategy: Comprehensive layers of government-level changes to historic practices resulting in legislation and guidance for lighting design and application for the whole Emirate.

Revised and enhanced statutory lighting standards were approved by Abu Dhabi Executive Council and publicly released in April 2017 as a single unified Abu Dhabi Emirate Lighting Manual PR-405, superseding all previous stakeholder-specific lighting documents and briefs. This 300-page lighting Manual is now the only standard to follow for all Abu Dhabi Public Ream and Street lighting projects.

This unified Manual followed the 2014 First Edition publication of a new 500-page Lighting Reference Handbook has provided in-depth guidance on every aspect of lighting design for the Emirate’s roads and public realm. A Handbook that is tailored for a region using only local imagery and examples and with over 50 different nationalities, a publication needing also to be understood by a variety of levels of English speaking competence and for all stakeholders and all members of the design chain. The Manual and supporting Handbook, see Figure 1, together now cover all aspects of lighting design for public projects for the future, and future revisions will be considered annually and undertaken by appointed Working Groups from all stakeholders under the supervision of the Abu Dhabi Quality & Conformity Council (QCC).

Figure 1. Abu Dhabi Lighting Manual Ver.1.0 & Lighting Handbook 1st.Ed.

Abu Dhabi has worked on its Lighting Strategy for over 7-years, with all public realm projects since 2011 designed, approved and installed in line with the new policies arising from the initial Strategy and the series of evolving standards culminating in the 2017 unified Emirate Lighting Manual. This paper shows how the strategy work was undertaken in regards to street lighting; why all public lighting needs to be more than just technical lighting indices and instead understood and re-considered from a human perspective and how this knowledge can not only maximise sustainability, but improve driver and pedestrian safety at the same time.

Within the 2017 Lighting Manual there are a number of revised and new mandatory requirements which together have harvested Abu Dhabi’s reputation as an international leader for public lighting, most notably for the ground-up re-appraisal of street lighting. Below are some of the major revolutionary inclusions within the new Manual, the reasons behind the key street lighting aspects will be explained within this paper:
• Design lighting levels for all road categories have been reduced significantly from historic practices and are now among the lowest in the world
• Light qualitative aspects are embedded in all calculation requirements
• LED (or proved equivalent) technology is mandatory for all public lighting
• Full ‘smart’ controls have become statutory for all new or retrofit street lighting projects
• Solar lighting material and performance specifications have been introduced after the results of 4-years of research and local data accumulation
• Light pollution legislation for the first time has become a requirement for planning permission and project design review submissions

Abu Dhabi City Municipality (ADM), under the Department of Municipal Affairs & Transport (DMAT) led all the work on the Lighting Strategy and policy document publications. But it has not just been about the new policies and documents: ADM has been able to use initial project installations to record operational data, installation performance and importantly public reaction. This has proved an invaluable tool in verifying the initial estimates are being met and most cases found to have been improved upon. As each year passes, the accumulation of more data gave the confidence to be able to recommend even more improvements to the original Strategy, including potentially more dramatic increases in sustainability and energy saving benefits. But the safety and quality of the lit urban environments is also being further enhanced in tandem.

2 LIGHT, COLOUR AND HUMAN PERCEPTION

Before one looks at lighting, one needs to understand some fundamentals of how we see the world around us that might be surprising:

• Human beings do not just ‘see’ with their eyes
• And we cannot ‘see’ light itself
• What we see is not precisely determined by any SI-unit technical lighting levels, Lux (illuminance) levels particularly
• Colour is not an absolute attribute of an object/surface; it is just a translated human perception
• There is no such thing as the colour white nor white light
• Black is not a colour either, just the absence of light

So what is light and how do we actually see? Ancient Greek philosophers such as Plato thought that light shoots out of the eyes to touch objects, much in the same way that we reach out and touch objects with our fingers. Another notion was that objects give out waves, like ripples of water from a dropped stone, and these waves radiate out and hit our eyes with the shape and colour of the object. Only much later when images were physically observed being projected onto the back of the eye (upside down; this will be explained later) did these initial ideas go away and science had a basis on which to explore how things actually worked.

Figure 2. The human visible spectrum and CIE 1931 Chromaticity Diagram mapping the colour relationship
How do humans then really see the world? Well, we have evolved over millennia to be able to respond to a particular narrow wavelength-band within the entire electromagnetic spectrum that stretches from cosmic rays to radio waves. Sitting sandwiched between infra-red and ultraviolet, human eyes have the ability to react to wavelengths from approximately 780 nano-metres (nm) to 380nm. See Figure 2, also showing how wavelength relationships were mapped by the Commission internationale de l'Eclairage (CIE) in 1931.

We respond to these wavelengths using approximately 120 million different types of chemical receptors, termed photoreceptors, contained within the back of each eye on the retina and these are concentrated around a focal point; the fovea, which is aligned behind the eye’s optical control devices (the iris, pupil, lens etc.). The photoreceptors, then funnel these reactions down through around 1 million fibres bunched into each optic nerve to send a constant stream of electrical signals to the brain. 1 million signals, or pixels if one equates this as a digital photo, is not actually that high a resolution; similar only to 1250x800 would be a similar digital photo resolution which most smartphone cameras easily exceed today. The brain however fills in the gaps to form the sharp extremely high resolution image that we ultimately perceive. It is therefore the brain and not the eyes alone that essentially ‘informs’ us what we are seeing by constantly processing these pulses as it feels is correct. It is termed the eye-brain mechanism: (Gregory, 1998) See Figure 3.

The eyes are thus essentially just our input sensors and on their own are merely the first element in the complicated process of seeing. After a combination of optical and chemical eye changes to adapt to the surround, the brain processes this visual stimuli based on the electrical signals received, but it then translates this information and fills in the spaces using a combination of pre-condition, expectation and experience, it tries to make sense with what it is presented and has rules and knowledge which informs its translation. As the natural world is both three-dimensional and moving, static two-dimensional and/or incorrect visual stimuli received by the brain can be ‘directed’ by the brain and converted to what it normally experiences.

In the daytime we end up having an accurate picture of our surrounding environment as the sun high in the sky washes the world in a bright all-encompassing and understood manner of light and shade, but at night when one has only artificial lighting as the eye’s reference source of illumination, if the stimuli is not typically rendered, or there is too much and/or too fast movement with unnatural contrasts, or not having any movement at all even, this can overload or confuse the ability of the brain’s processing and as result can present an incorrect translation of what is actually there. Add into the mix the height and brightness of the visual stimuli; too much intense source-light will cause the eye to strain, be disabled or worst case damaged, too little and photoreceptors cannot chemically react sufficiently to anything in the field of view.

Light enters the eye through the cornea and because this part of the eye is curved, it bends the light, creating an upside down image on the retina. The brain already makes this initial necessary translation, and as there are two eyes does this three-dimensionally, but this is just the start of the work it does to translate the world correctly. Optical illusions interestingly can be considered as excellent examples of deliberate confusion of the eye-brain-mechanism at work. Such illusions are designed to overload or confuse the brain’s historic rulebook of what it should make from the stimuli it is presented with. This is why, for example, incorrect colours or contrast changes are apparent even after one is presented with proof to the contrary, or why apparent movement occurs within flat static patterns when the brain is trying to make sense of what it expects should be a three-dimensional world, or completely obvious parallel lines appearing to be anything
but that when patterns are placed behind them. Refer to Figure 4. for some examples of the brain manipulating one’s eyes:

Figure 4. Clockwise from top left: Parallel horizontal lines distorted by high-contrast background pattern, Flashing white dots caused by a dark surrounding grid, Same colour of pink changed by shade of bordering squares, Same colour red and green rings distorted by background colour, Old woman or young lady?

The most relevant optical illusions in relation to the main subject of this paper; namely safer street lighting, are when the brain is deliberately bombarded with too much information; having many objects, along with high light/dark contrasts, and the image has additionally some moving elements. When these types of illusions are experienced, the brain has so much to process it prioritises and narrows the field of processing to the central portion and peripheral vision is temporarily suspended. The result can be parts of images to the sides (typically strategically placed dots or shapes) just appear to vanish completely or randomly flash on and off. Whilst not possible to demonstrate in this paper, this will form part of the Congress presentation.

After sundown, streets against a night sky become high contrast environments, they have many objects and forms of artificial lighting within the field of view and a lot of movement of these in various directions, plus the observer is moving as well. Potentially all the attributes of an optical illusion are provided. But just imagine what might be the result if the brain gets overloaded and shuts down what drivers can see at the sides of their vision or eyes become strained or disabled, even for just a few seconds?

Briefly colour needs a mention as this, along with light, is not a straightforward subject when it comes to human perception. Humans do not see light; we receive wavelengths that are reflected from objects and surfaces. Depending on the quality and spectral makeup of the light source (the sun is as perfect as one can get on this count), and the material absorption/reflection properties of surfaces/objects, the light reflected into our eyes leaves these surfaces/objects with a specific resultant wavelength and it is this wavelength that we translate to perceive a colour our eye-brain mechanism assigns to it.

Even luminaire light sources are only visible as they have phosphors, gas, filaments and or optics which are essentially surfaces themselves that emit light. As emitting sources such as the sun, artificial lamps or, in our case, street lighting luminaires tend to be very powerful, none can be looked at directly without causing glare issues; temporary distraction, disability or in the case of the sun and other very bright sources, eye damage to
the cornea which could be permanent. As such what we need is these powerful sources to be suitably shielded from direct view and what we comfortably receive is the resultant light reflected from surfaces. Figure 5. Shows the viewpoint under which a typical glare calculation is based: (Phillips, 2002)

Figure 5. Example of sodium street lighting with excessive direct glare even well above the horizontal plane and diagram showing how driver glare threshold limit is assessed in relation to street luminaires

There is no such thing as white light; none of the visible wavelengths are white, only millions of colours from red through to blue. But when all the wavelengths are present, the combined affect is to see white. It is termed additive mixing. The sun as mentioned emits every wavelength and as such is considered the perfect (reference) white light source, which means light reflected from objects and sources from the sun is not missing any wavelengths and thus the colours we receive into our eyes and the eye-brain perceives are the benchmark for human colour perception.

But current artificial light sources cannot mimic the sun, these all target basically only three primary colours to achieve white light; Red, Green and Blue. With the exception of monochromatic low-pressure sodium, all commercially viable energy-efficient forms of artificial lighting, such as fluorescent, discharge and LED technology use gases, metals and/or phosphors to emit peak light in these three wavelengths which combine to produce a tuned pseudo white light; a compromised quality of white light, which is lacking, to varying degrees, certain other colour wavelengths in between the three peaks. As such, the light reflected into our eyes from objects and surfaces is also missing the input of these coloured wavelengths and thus the surfaces and objects can appear to be rendered different colours as compared to how they would look in daytime, and very deficient in certain colours: (Pritchard, D.C.). See Figure 6. which shows the qualitative attribute of different sources, called Correlated Colour Rendering Index (Ra)

Figure 6. Example of how coloured objects rendered from varying Ra traditional artificial sources including their respective typical spectral outputs (LED is typically similar to Tri-Phosphor Fluorescent)

But this is not all bad as our needs for perfect colour accuracy are task led and the needs for a museum say are different to the needs of an office or for the lighting of streets. The former obviously needs to have the best possible natural and artificial lighting solutions available, whilst for the latter the needs are more basic and driver and pedestrian safety is the key rather than absolute perfection of rendered colour perception.
The final part of the equation is that the 120,000 Million photoreceptors found in each human eye are split into two distinct types: Rods and Cones (named so due to their general shape viewed under a microscope). Rods detect only contrast; light and dark, they are very sensitive and operate only when there are very low levels of ambient light, essentially providing us with basic monochromatic vision when there is just a tiny amount of light around, such as on a moonlit night. This is termed Scotopic Vision.

However rods become redundant once light levels rise and then the cones start operating. Cones, of which there are far more found on the retina than rods, come in three chemical forms that react to red, green and blue wavelengths specifically. The varying degree of reaction of these is translated by the brain as specific colours. This is termed Photopic Vision. So one can appreciate that artificial lighting sources that all work on emitting red, green and blue wavelengths are not that compromised after all if humans have a predisposition to receive and transpose this information using three similar peak-wavelength receptors. Figure 7. shows the relationship of cone and rod receptors in human eyes.

Why do we only have these three colour receptors when the sun emits a multitude of additional wavelengths? The reason is this is all we need and evolved with for necessary vision as hunter-gatherer mammals. We are of course not the only species on the planet and the other wavelengths are all used in various different ways by plants and other animal species; insects, birds, reptiles, even bacteria.

What has been discovered and increasingly explored over the last three decades though is that not all human cone receptors are as efficient as others; red is the weakest and green just pips blue as being the most light-sensitive; around 510 to 555nm typically. What this means is that humans can respond to green and blue light around these wavelengths at lower levels than we do red and as such artificial light with more green and blue found within them can actually make surfaces and objects appear brighter to us even at the same SI-unit lighting levels as that achieved with sources deficient in green and blue. See Figure 8.
3 STREET LIGHTING

Taking onboard all of the previous information covered in Section 2, the right artificial lighting design should then be about providing appropriate human visual environments for the eye/brain to correctly process the task at hand. For driving, this is to provide the optimum balance of brightness-uniformity, minimised glare, peak-response colour temperature and light quality and then tailoring these for particular road categories, road surface types and travel speeds expected. Whether this is with LED, Sodium or any light source, the aims must be the same.

Why do we need street lighting at all? Many countries have roads, whether by deliberate local standards or from lack of infrastructure/budget, that do not have any street lighting and whilst headlights alone can provide some level of safe vision for drivers, the ability to detect hazards at distance and be able to react at speed, plus eye strain and tiredness issues are a big safety concern. In addition any areas with pedestrian activity would be a compounding risk for both the pedestrians and drivers. The other major and perhaps unique factor for the region is that the needed tinted windows for daytime sun comfort conversely reduce visibility at night, especially driver rear and side viewing ability where his or her headlights are not pointing. Where street lighting can be provided it is always going to be the safer option for all road users.

Back in the 1960s the first real attempts by street engineering societies at establishing road lighting standards arose in the US and around Europe. Certain aspects of the needs for drivers were understood and centered on providing base levels of luminances or illuminances appropriate for a size, speed and use of different road types. However artificial light source technology at the time was really limited to either low-pressure or high-pressure sodium fixtures and concentrated on standardising basic reflector distribution Types. Criteria for covering the qualitative aspects of lighting or managing glare and light pollution were severely lacking. Figure 9. Shows examples of the early North American Standards approach.

<table>
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<th>Roadway Lighting Criteria</th>
<th>Roadway</th>
<th>Area</th>
<th>$E_{avg}$</th>
<th>$E_{avg}$/Lmin</th>
<th>$E_{avg}$/Lmin (cd/m²)</th>
<th>$L_{avg}$/Lmin</th>
<th>$L_{max}$/Lmin</th>
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<tr>
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<td>Med</td>
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<td>4.0</td>
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<td>6.0</td>
<td>0.50</td>
<td>6.00</td>
<td>10.0</td>
<td>0.4</td>
<td></td>
</tr>
</tbody>
</table>

Source: ANSI/IESNA RP-5-00

Figure 9. ANSI/IESNA (Illuminating Engineering Society of North America) Street lighting level standards and key to fixture Type classifications

Low-pressure sodium was the cheapest option, as had the longest life and the most light-output to power ratio (lumens per watt), however they are monochromatic as demonstrated in the previous section, so only emitted a yellow wavelength. This is turn means that only yellow light hits surfaces and objects and humans as a result only saw the world in shades of yellow, greys and black (demonstrated in Figure 6.). Yellows would appear yellow, but everything else; red, blue, green etc. would be rendered as shades of grey. High pressure sodium emits more wavelengths, but predominantly in the yellow, orange, red spectrum and is not as long-lasting or cost efficient as its low-pressure cousin.

One thing which does differentiate street lighting design from this time, as can be seen in the table in Figure 9, from nearly all other forms of lighting design is the use of road surface Luminance (Cd/m²) in standards as opposed to purely illuminance (Lux (or foot-candles in the US)). Luminance is a measure of the amount of light received by a typical observer and in the case of street lighting this is from a specific point (typical eye-level of a driver 60m away is the measure) after reflected from the road surface and is thus a much better comparison because it takes into account the colour and absorption/reflection characteristics of a specific material. Illuminance on the other hand is just a measure of light falling onto a surface/object and disregards the material itself.
To explain this difference; if one calculates or measures an illuminance level of say 500 Lux (a typical office desk standard for workplaces) on a white desk or dark wood desk it would be the same SI figure, however we would of course see the white desk as far brighter than the dark wood one and this is because Lux does not account for the greater losses of reflected light from the dark wood desk. The luminance of the desks as observed would on the other hand be vastly different even though the illuminance is the same. So Lux is flawed and can be misleading if one wants to know the true visual situation. It is however far easier to calculate and verify with site measurements, which is why it is still used in most other forms of lighting design today. Using Luminance for street lighting on the other hand means that if a street surface is light concrete or dark asphalt, this is taken into account and the design ensures the apparent brightness at the observer is always the benchmark.

So road lighting classification systems were born which described roads as numbered classes, for which each had set luminance design targets and luminaire have class Types. These were stated based on sodium lighting as were the only options at that time. There has been some development over the decades in understanding of needed contrast ratios on the road surface, glare limits of the luminaires for drivers and appropriate lighting levels to the sides of roads. But generally today’s engineering-led standards for street lighting, based on sodium sources, might be surprising to hear are little different to those from the 60s. Whilst these are all sensible and verified over the years, what has been disappointing to see is that stakeholders and manufacturers do not understand what is behind these standards and lack the ability or inclination to challenge or expand tabular matrices where things might be able to be looked at in a more tailored manner? The luminaire manufacturing world too appears to define good and bad street lighting in terms of one technology over another, wattages, maximum efficiency, energy saving controls and price etc.

4 LIGHT EMITTING DIODE (LED) TECHNOLOGY

LEDs generally have a long life and may last up to 200,000 hours in some commercial applications. LEDs generally emit light in a relatively narrow band so that most LEDs produce light that is a saturated colour. White LEDs are most commonly made by using a blue chip and placing a coating of phosphors around it. The phosphors are a tuned recipe to react to the blue source and produce the additional green, red and other colour spectrums that are needed for a quality white light to be emitted.

The long life of LEDs is possible because they are a solid state technology and thus have no vacuums, gases, reactive metal amalgams or filaments, all of which degrade and are weaknesses and the cause of far earlier failure rates found in traditional sources such as discharge, fluorescent or filament lamps. The very small size and directional nature of LEDs also means their light can be focused far more precisely and with lower light losses within luminaires.

Recent LED development has also resulted in very high efficacies (the amount of light output per unit of input energy), which on top of much better high temperature tolerance, a drawback in earlier generations, has resulted in LEDs becoming the dominant light source in every sector of lighting, including street lighting applications. They are also available in a whole range of different white colours (Correlated Colour Temperatures: CCT) which are more efficient with the cooler white colours as compared to the warmer ones, primarily as there is more of the chip-origin blue emitted from the phosphor coating.

5 RESULTS: THE ABU DHABI SOLUTION

Making Human beings as the primary focus on street lighting and how we actually see, our needs physically and the light quality parameters seems increasingly to have been ignored, even though we now have a far greater understanding of these aspects in other sectors. This has not been the case in Abu Dhabi and we have ensured every aspect of street lighting was challenged, benchmarked and as required changed to ensure Abu Dhabi has an optimum set of lighting standards for the years ahead.

Current street lighting matrices can indeed be translated to how they are intended to address some of the needs for the human eye-brain mechanism. The below are the primary parameters used for which target levels are always attributed and the translation of what they mean for drivers:
- **Luminance levels (Cdm\(^2\))** = The brightness of the road surface needed at the driver’s eye to allow correct reaction and perception for the speed and type of road.

- **Uniformity Ratio (Uo)** = The minimum to average ratio of luminance across the whole road surface to ensure contrast allows ability to detect whole visual field objects such as adjacent traffic movement.

- **Longitudinal Uniformity (Ul)** = The minimum to maximum ratio of luminance along each lane’s centre-line to ensure contrast allows ability to detect obstructions/objects in the immediate direction of travel in time to safely react.

- **Threshold Increment (TI)** = Matrix for direct street light-source glare minimum level at driver observation level and distance to ensure no distracting or disabling effect to the eye.

- **Surround Ratio (SR) (will be superseded by Edge Illuminance Ratio (EID))** = Percentage of illuminance as compared to the road surface illuminance at the sides of the roads to ensure drivers can react to pedestrians/traffic entering the roads.

- **Conflict Areas (ramps, roundabouts, intersections)** = Usually doubled illuminance levels for transition areas to or from the roads in which the driver needs to account for potentially more conflicts with other road users and/or from multiple directions.

- **Light Fixture Source Colour Temperature (CCT)** = The colour temperature of white light at source from warm to cool. For example Sodium is very warm around 2000Kelvin(K), whilst fluorescent and LED fixtures usually range from a warm 3000K up to very cold 6500K. Is used currently by most only for the consequential look and feel of street lighting and streetscapes.

- **Light Fixture Source Correlated Colour Rendering Index (Ra)** = Measure of the degree (percentage) to which the light source has the ability to render an object’s colour correctly as compared to a 100% reference source. Uses 8 pastel test colours only and is flawed as it both ignores bold test colours as well as the reference source used being a tungsten lamp and not the sun. New CIE and IES standards are drafted to improve this rating of light sources, but as yet have not been internationally agreed or adopted.

Apart from these target figures, little else is considered in designs and the process has been reduced to a mathematical lighting calculation (between two poles only!), with each benchmark a pass or fail on an checklist and 2-Dimensional approach for all projects and alas nothing changing with use of LED. Figure 10 shows projects with sodium and LED where design has fallen short of proving a correct visual environment.

![Figure 10. Excessive contrast with Sodium in pre-LED LA and LED only lighting half a Chinese highway](image-url)

Even more prevalent here in the Middle East is an all too common misconception with artificial lighting and especially with street lighting that more light is better and that brighter lighting means safer roads. Real-world observations of sodium street lighting installations and indeed LED, that do albeit meet locally adapted...
international standards, have often ‘looked’ awful to people as standards have always been just about minimums and average. Much of this is down to a lack of correct lighting design, but for sodium also due to the historic drawbacks of such intrinsically large discharge lamps; traditional sodium fixtures do produce a lot of light, but the large size of the lamps makes it difficult to focus and spread that light across the road surface evenly. The result is much of the light misses where it should go (i.e. the road surface and just the immediate surround) and they additionally create hot-spots under the fixtures and harsh shadows in between the poles which, although within minimum international best-practice uniformity limits, does not promote a comfortable visual environment.

But with LED these issues should not be happening, however the fact we seem to be having so many negative reports in the news on LED street lighting is an indication that they are, and in fact lighting design seems to be completely forgotten with some LED changeover schemes with only energy the focus and this is unforgivable.

The solution to these issues has been too often the practice that the roads can be improved simply by raising the lighting level higher and higher. It is in fact the opposite; all that happens by raising the light levels by using bigger sodium or other discharge lamped fixtures, or more recently equally powerful LED fixtures, is make the hot-spots brighter and, by contrast, the shadows darker; often breaching the minimum uniformity limits, i.e. making the roads actually worse and more dangerous visually. As levels have been raised and raised over the years around the region, this check on uniformity appears unfortunately often to have not been part of their retrofitting or Standards-revision exercise.

Why focus on higher lighting levels when better light distribution and thus improved uniformity would allow lower lighting levels? Why do we need the same lumen output from optically better LED fixtures when replacing sodium for example? What about the light emitted from LED as far as spectral content and quality and what is the impact, either beneficial or detrimental, of the different white colours afforded to us? Why do we need to follow historic international road lighting classes just because a document states we should? Abu Dhabi challenged all of these aspects and more in the Lighting Strategy work undertaken. Figure 11 below shows the 2010-2017 changes made by Abu Dhabi and how this is compares internationally and regionally:

![Image of lighting strategy comparison](image-url)

Figure 11. In 7 years Abu Dhabi has, in two stages, dropped road levels to amongst the lowest in the world

Abu Dhabi looked at the higher blue and green content of white LEDs used in street lighting fixtures and understood that this enables the human eyes’ peak sensitivity to react and ‘see’ at much lower comparable
levels to the red, orange, yellow wavelengths dominant in sodium lamps. As such Abu Dhabi both implemented the mandatory use of LED fixtures at the same time as actually lowering all lighting levels from 40% to 70% across all road categories. We ensured light quality came before calculations and optical performance for light distribution was maximized to ensure better light uniformity on the road surface and light pollution reduced outside. Figures 12 and 13 below shows how correct LED design to the 2017 Manual can visually improve roads even though the luminance levels are over half of that of poor existing practices:

Figure 12. Mockups showed stakeholders that the lower luminances work combined with the more human-sensitive whiter light of good LED fixtures, tighter glare control and better road-surface uniformity standards.

Figure 13. The new 2017 Abu Dhabi Lighting Manual’s table of street lighting levels which are comprehensive and mandate the safest and most efficient lighting standards in the region: (Government of Abu Dhabi, 2017)
6 CONCLUSIONS

Whilst other counties inside and outside the region have also begun a move to LED for street lighting, they have done so by not fully considering the human-centric aspects. Whilst elsewhere typically LED street lighting is saving 30 to 40% on energy consumption, the visual environments have not necessarily been improved upon and in many cases made even worse. Abu Dhabi, through considering all the human aspects and not just technical, is achieving an unprecedented 70 to 90% energy saving on projects whilst also improving the lighting for people. The Sector Internal Street 0.4Cd/m² mockup shown in Figure 12 for example replaced original 400W Sodium fixtures with just 41W LED luminaires. This policy has been done to ensure the necessary move to energy saving and sustainability for street lighting infrastructure is not only undertaken with optimum solutions, but this does not come at a cost of safety and quality and instead is done to improve safety, public wellbeing and create and vastly improved visual environment concurrently.

Abu Dhabi’s Strategy and new unified Lighting Manual which embeds all these refined street lighting requirements and accompanying guidance Handbook have to date won ten regional and international awards for public lighting and are becoming the benchmark for changes internationally.

Every country can achieve similar vastly improved savings on street lighting by improving uniformity, reducing glare and actually lowing lighting levels and not keeping them the same or raising them when moving to LED. The whiter, more reactive light to human eyes, and better optical control of quality LED street lighting fixtures combines to allows safer, better more sustainable roads for the future.

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Abu Dhabi Executive Council, Department of Municipal Affairs & Transport, Abu Dhabi Urban Planning Council, Al Ain City Municipality, Al Dhafra Municipality, Abu Dhabi Quality & Conformity Council,


REFERENCES


A Framework for Assessment of Bridge Barrier Rehabilitation Requirements on Existing Overpasses

4- Towards Zero Fatalities and Serious Injuries
4.3 Safer Streets by Design

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KEYWORDS:
Test Level Selection, Rehabilitation, Strengthening, Traffic/Bridge Railing, Yield Line Analysis

ABSTRACT:
Traffic safety on many longstanding bridges is getting worse than ever, leading to many fatal accidents. Such structures may not comply with the recent, more demanding road design controls which reflect current vehicle performance and human behavior.

The main objective of this study is to develop and implement a framework for assessment of the need to replace old/existing bridge barriers in order to achieve safer test levels. Such an assessment is of particular importance for bridges that were executed before the 1990’s, posing additional challenges to incorporate the impact of such a bridge barrier upgrade.

Given that there is no standardized methodology for upgrading bridge barriers associated with the level of safety that needs to be attained, the paper identifies the technical challenges and proposes specific criteria to alleviate them. As such, the paper describes a proposed framework that takes into account the structural capacity of bridges to withstand vehicle collision loads due to the upgrade of the bridge barriers in addition to addressing other essential factors such as test level selection, value engineering, construction methods, cost, traffic volumes, structural aging effect, and traffic diversion needs.

The proposed framework was applied to twenty-three (23) interchanges which were selected along the most congested roads in the city of Riyadh-KSA, including the First Ring Road and Makkah Road.
1.0 Introduction
For an existing bridge having an older safety shape railing with reinforcing that doesn’t meet current standards, the first step should be to determine if it can remain. If the existing railing was found to be substandard, strengthening or replacement will be required. The replacement of barriers on existing bridges offers a number of individual challenges during the planning, design, construction, and service life phases. Special attention is needed in both the design and detailing stages of the replaced barrier in order to minimize construction and maintenance problems later on. Detailed information should be obtained prior to deciding on how to proceed with the design of the new barrier. Some useful sources which may be used are: (1) bridge site submittal, (2) structure maintenance and investigations records, (3) preliminary site investigation visits, (4) as-built construction drawings, and (5) photo log from a driver’s perspective.
When the existing barrier is deemed obsolete or substandard, a new barrier needs to be selected, designed and installed. The design of newly constructed bridge railings must conform to the requirements of Section 13 of the AASHTO LRFD Bridge Design Specifications (2014). This specification gives geometric and strength requirements and also describes crash test levels. There are six levels of service and testing depending on vehicle size and speed. The most commonly used barriers are made from either concrete and/or steel. In the case of concrete barriers they are usually fixed such that when struck, deformation is small. Hence they are commonly referred to as rigid concrete barriers. Steel tubing can be fixed to the top of concrete barriers to provide extra height in order to prevent vehicles with a high center of gravity, e.g. trucks, from rolling over the top of them.
Selection of a new barrier and method of installation will depend on many factors including the thickness of the existing slab, length of deck slab overhang, and test-level of the barrier. Concrete safety shape barriers are an economical retrofit design and low maintenance if the structure can carry the added dead load and if the existing curb and railing configuration can meet the anchorage and impact forces needed for the retrofit barrier. Concrete barriers utilize a single-slope or safety shape (e.g. New Jersey or F-Shape) to redirect vehicles while minimizing vehicle vaulting, rolling, and snagging. Figure 1 shows five common shapes of concrete barriers. Typical steel reinforcement in a NJ-shape parapet with vertical back face is shown in Figure 2.

![Figure 1: Concrete Barrier Shapes](image-url)
An existing bridge parapet-and-deck-overhang system may not meet current standards in two ways, as shown in Figure 3: (1) vehicular collision forces, and (2) gravity live load (including the dynamic load allowance) and dead load effects.

2.0 Literature Review

It is worth to start investigating all effort and requirements on bridge railing as retrieved from common and international codes such as AASHTO LRFD 2012 where it defines the barrier specifications according to the Test Level-TL level for any road or bridge, which depends on a range of factors affecting the level of safety, including:

- Speed vehicles on the road or bridge
- Percentage of trucks passing by
- Number of lanes and direction of traffic (one direction or two directions)
- The path of the bridge (straight or curved), and its height and inclination
- Obstacles to meet vision requirements
- The possibility of passing large trucks (locomotive and trailer, oil tankers, etc.)
- The presence of sidewalks at the checkpoints

The above-mentioned manual identifies six levels of safety as below:

- **TL-1**: Test Level One—taken to be generally acceptable for work zones with low posted speeds and very low volume, low speed local streets
- **TL-2**: Test Level Two—taken to be generally acceptable for work zones and most local and collector roads with favorable site conditions as well as where a small number of heavy vehicles is expected and posted speeds are reduced
Among the early published literature on bridge parapets is the work of Nordlin et al. (1971), in which full-scale field tests were conducted on five 99 cm high concrete barriers topped with steel tube railing. The impact tests showed that the parapet is capable of resisting 22 kN force applied at speeds up to 101 km/hr and at angles of 7-25 degrees with the barrier. A taller parapet with an overall height of 137 cm was crash tested by Hirsch and Arnold (1983). The barrier was able to restrain and redirect a lateral impact by a 363 kN van-type tractor-trailer at a speed of 80 km/hr and 15-degree angle impact. In 1990, Buth et al. (1990) tested one steel beam-and-post design and three concrete parapets by subjecting them to 81 kN lateral force at 15-degree angle. While the concrete parapets demonstrated no structural distress, the metal railing showed some damage in the bolted rail-to-post connections. Saldana and Mendelson (1992) assessed a 100-year old bridge in Edinburgh and found the parapets to be below standard. Special refurbishment techniques involving materials not commonly utilized in bridge construction were utilized in order to provide an aesthetically acceptable solution while complying with the specification. In a joint research program, Alberson et al. (1995) and Menges et al. (1995) studied performance levels 1 and 3 bridge railings. For performance level 1, two bridge railings were tested; the Oregon side-mounted steel railing and the BR27D concrete parapet with a metal beam-and-post railing mounted on top. For performance level 3, 107 cm high F-shape bridge railing and concrete parapet were utilized. In all cases, the railing performed acceptably. Faller et al. (2000) tested two bridge railings and transition systems for transverse timber deck bridges. One system was built with glulam timber components and another was configured with steel hardware. Overall, 8 full-scale crash tests were conducted, and the bridge railing and transition systems were found safe according to the relevant standards. Naito (2005) investigated the anchorage of reinforced concrete parapets on glass fiber reinforced polymer bridge decks. All but one of the 6 FRP decks performed exceptionally well by remaining undamaged up to the ultimate capacity of the parapet. The tests showed that the use of headed reinforcement embedded in the bottom deck flange is a better than utilizing reinforcement-to-resin anchorage transfer mechanism and external steel brackets can provide adequate strength and ductility when repairing damaged parapets on existing bridges. Clubley et al. (2007) evaluated the lateral load resistance and distribution of bridge parapets in Southern England. Field tests and theoretical studies showed that only the shear links immediately surrounding the anchorage post are effective in resisting the load, and a simple load dispersal angle of 65° is reasonable for assessment calculations. The behavior of high performance fiber reinforced concrete precast parapets under quasi-static and dynamic loading was addressed by Charring et al. (2011). Results of the quasi-static tests showed that FRC parapets have comparable strength and ductility to conventional parapets. Quasi-static tests conducted following the dynamic tests demonstrated residual strength equal to 75-100% of their original capacity. The developed finite-element model was able to acceptably replicate the strength, stiffness, and failure mode of the parapets. Williams and Boyd (2011) developed designs guidelines and details of two retrofit steel railings that use adhesive anchoring systems. The refurbished system meets the strength requirements of Test Level 4. Full-scale strength testing data available from the literature were used to develop the two retrofit systems. Etoh and Thanh (2014) studied steel railings installed on concave-and convex-curved bridge parapets using experimental tests and finite element analyses. For concave road curvatures, it was found that the impact angles between the vehicle and the railings were larger than assumed in the current specifications. The results for concave-and convex-curved railings were compared against each other as well as with the results for straight railings. Recently, Kalabon et al. (2014) investigated the causes of cracking in concrete parapets installed on bridges, including properties of the concrete mixture, construction methods, joint details, composite action with the deck, and durability of the concrete and reinforcement. It was found that the use of glass fiber reinforced polymer rebars and making deeper saw cuts through the reinforcement for the control joints are the best solution to prevent early age cracking in the concrete.
3.0 Study Methodology

A methodology is proposed in figure 4 below showing the general framework bridge barrier rehabilitation. Rehabilitation of an existing barrier starts with an assessment of the existing barrier in terms of structural distress and weathering in addition to the assessment of the required test level on existing railing. The credibility of both assessments, i.e. as is the case of most rehabilitation projects, is directly affected by data collected and comprehensiveness thus ensuring both qualitative and quantitative appraisals. Subsequently a comparison of the above assessments shall lead to two alternatives denoted as compliance or non-compliance of the bridge railing with respect to the required Test Levels.

- If non-compliance, the selection of barrier type shall influence the installation of the new barrier on the existing bridge; nonetheless, the structural impact on the existing bridge is also worth investigating noting that it is most probable that such amplified impact resistance is not catered for in the current bridge design. Based on the above, the proposed solution shall comply with both test level requirements and bridge structural capacity. If the existing reinforcement in the deck slab is not adequate, then corrective measures in the form of additional steel are needed to ensure safety and acceptable performance.

- If the required test level is compliant with the existing bridge railing, a more thorough assessment on site conditions shall follow for corrective maintenance in case the barrier is already damaged and/or partially weathered in some areas and then to proceed with preventive maintenance to ensure all serviceability requirements.

Figure 4: Study methodology outline

4.0 Assessment of Existing Barriers on Structures

Current design requirements in the AASHTO LRFD bridge design specifications (2014) for the new barrier are based on the yield line theory, as shown in Figure 5 below.
Figure 5: Yield line analysis of the barrier due to vehicle collision forces

The assessment shall consider independently the parapets located within wall segments (interior region) and those located near end of wall segment (end region). The capacity of both cases differ based on the yield line theory, the interior region provides higher capacity as compared to the end region keeping other things equal. For the interior region, three yield lines are required for the barrier to fail, two lines producing tension on the inner side of the parapet and one line yields on the outer face of the parapet. For the interior region, only one yield line producing tension on the inner face of the parapet is required for the element to fail under vehicle collision forces. In addition, the distribution length mechanism differs between interior region $L_{ci}$ and end region $L_{ce}$ ($L_{ci} > L_{ce}$).

Various components contribute to the structural capacity of the barrier:
- Concrete barrier dimensions and compressive strength;
- Barrier Reinforcement, both vertical and horizontal (flexural capacity of the barrier about its vertical and horizontal axis);
- Beam/thickening at top of barrier (cap beam), if any;

The overhang deck slab will receive the load effect from the barrier in the form of tension and flexure, as shown in Figure 6. Therefore, the reinforcement that is present in the existing slab must be able to resist the additional forces due to vehicle collision.

Figure 6: Forces induced from the barrier into the deck slab overhang due to vehicle collision

In addition to designing the deck for dead and live loads at the strength limit state, The AASHTO-LRFD specifications require checking the deck for vehicular collision with the railing system at the extreme event limit state. The equivalent strip method is an approximate and practical method and is based on the following:
- A transverse strip of the deck is assumed to support the truck axle load;
- The width of the strip for different load effects is determined;
The truck axle loads are moved laterally on the bridge overhang to produce the moment envelopes. Multiple presence factors and the dynamic load allowance are included. The total moment is divided by the strip distribution width to determine the live load per unit width;

- The loads transmitted to the bridge deck during vehicular collision with the railing system are determined;
- Load factors are then applied as per code requirements to determine design factored loads;
- Reinforcement is calculated using conventional principles of reinforced concrete design;

Bridge deck/overhang capacity is then checked versus applied straining actions to verify whether existing sections have adequate capacity to resist applied loads or whether deck strengthening is required (refer to section 5.1.2).

### 5.0 Railing System Test Level Selection Criteria for Bridges

A group of international scientific references has been reviewed to compare and select criteria for assessing the appropriate level of safety for each bridge of study. The following references provide some appropriate foundations, albeit some lack of information as stated previously:

- AASHTO LRFD 2012-6th ed
- Roadside Design Guide 2002-AASHTO
- PennDOT Publication 13M August 2009
- Other DOT Highway Design Manuals (i.e. NYC)
- NCHRP Report 350

Unfortunately, AASHTO LRFD (2012) does not involve any quantitative assessment on the test level selection criteria for bridges, but only refers to some qualitative assessment and the governing parameters are depending on except for the above-mentioned Pennsylvania Department of Transportation bulletin which contains a clear practical application for selecting the appropriate safety level in relation to several factors that can be identified and measured for each road as the following:

- Road Design speed
- Daily traffic volumes
- Percentage of heavy vehicles
- Type of road (single or double) and number of lanes
- Railing offset

This reference includes specific tables for each design speed (multiple of 10 km/h) as shown in table 1 below:
Table 1: Test Level Selection Criteria for Design Speed of 100km/hr-Taken from PennDOT Publication 13M, (2009)

<table>
<thead>
<tr>
<th>Site Characteristics</th>
<th>Adjusted ADT Ranges for Railing Test Levels (10^3 vpd)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Highway Type</td>
</tr>
<tr>
<td></td>
<td>Divided or undivided with 5 or more lanes *</td>
</tr>
<tr>
<td></td>
<td>Undivided with 4 lanes or less</td>
</tr>
<tr>
<td></td>
<td>TL-4 permitted for Adjusted ADT up to the number below, otherwise, TL-5 is required</td>
</tr>
<tr>
<td></td>
<td>TL-4 permitted for Adjusted ADT up to the number below, otherwise, TL-5 is required</td>
</tr>
<tr>
<td>Design Speed (km/h)</td>
<td>Truck percent (mph) Rail Offset (mm) (ft)</td>
</tr>
<tr>
<td>100</td>
<td>60</td>
</tr>
<tr>
<td>100</td>
<td>60</td>
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<td>100</td>
<td>60</td>
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<tr>
<td>100</td>
<td>60</td>
</tr>
</tbody>
</table>

Particularly, the adjusted average daily traffic is corrected based on three essential other factors referred to as Kc, Kg, and Ks in accordance to equation (1) herein:

\[
\text{Adjusted ADT} = K_c \times K_g \times K_s \times (\text{ECT-ADT})
\]  

(1)

where

- \(K_c\) = correction factor for curvature
- \(K_g\) = correction factor for grade
- \(K_s\) = correction factor for drop-off distance and understructure conditions

(ECT-ADT) = estimated construction-year total ADT for the highway (limited for 10,000 vehicles per day per lane if speed \(\geq\) 80km/h)

6.0 Evaluation of Bridge Railing Test Level

To evaluate the bridge railing test level, the current test levels for each of the existing bridge railings under study shall be categorized with reference to the design forces for traffic railings depicted in AASHTO LRFD (2012) specifically table A13.2-1. By calculating the existing railing capacity in terms of the maximum design forces that could be attained, the bridge railing can be easily categorized between test levels 1 to 6. Then it is also possible to determine the test level required for bridges and intersections according to the criteria explained in section 3.0 above. Accordingly, the types of barriers required will be determined according to the recommended testing level to be then examined and evaluated under the effect of its constructability and the maximum load capacity/tolerance of the existing bridge; thus, the selection of the most suitable technical solution in terms of barrier installation and bridge strengthening will be further assessed to withstand the supplementary design forces of the proposed traffic railings.

The choice of the new barrier and the method of installation depend on many factors, including:

- The thickness of the current bridge slab
• Length and thickness of the bridge slab
• The structural state of the slab, especially the bridge slab, including reinforcing steel, general arrangement, etc.
• The extent of design forces improvement due to the difference in the current test level versus the proposed one

It is most likely that existing overpasses which require improvement in terms of barrier replacement and rehabilitation were designed and implemented over 25 years ago, thus the design should have been complying with the LFD-AASHTO or equivalent rather than LRFD-AASHTO. It is therefore expected that the LRFD-AASHTO requirements will not apply to existing bridge designs, apart from the effect of replacing existing barriers with improved barriers involving a higher test level. The assessment of the impact of replacing existing barriers with specific safety standards is naturally based on the LRFD-AASHTO specification, specifically in assessing the bridge's ability to resist loads resulting from new barriers. The design requirements in AASHTO-LFD are based on the impact of the vehicle on the A side load of 44.5 KN, which is far below LRFD-AASHTO requirements (refer to AASHTO LRFD - specifically table A13.2-1) and will be detailed in the following sections.

7.0 Assessment of Bridge Barrier Rehabilitation
7.1 Proposed Solutions
7.1.1 Barrier Fixation
When replacing a barrier on existing bridge, care must be exercised to remove just enough of the existing railing and curb to accommodate installation of the new barrier. Fixation of new barrier on existing bridge deck presents many challenges to ensure that the connection between the new barrier and the existing bridge deck have enough capacity to accommodate applied factored loads as per new AASHTO LRFD requirements and adopted test level. Many alternatives are studied as shown in figure 7 and list below:

1- partial or total demolition of the bridge overhang to allow proper overlap between existing and new reinforcement as well as strengthening of the bridge element if deemed necessary;
2- installation of precast barriers whereby fixation is achieved by installing anchor bolts at predefined configurations;
3- drilling into the existing deck slab and installing steel reinforcing bars using epoxy adhesives

Pros and cons of various alternatives are elaborated in terms of time and cost of construction to mitigate the effect of upgrading works on any anticipated traffic interruptions. Accordingly, alternative 3 was found to be the best option allowing faster time of installation as well as providing a uniform barrier-deck connection in addition to a moderate cost as compared to demolition of existing bridge overhang. However, one main challenge was to check the embedment capacity of the new barrier reinforcement to ensure that no premature failure of the connection occurs at the interface between old and new concrete. Many factors are deemed to affect the final capacity of the barrier-overhang connection:

- Properties of existing and new concrete, mainly compressive strength;
- Tensile strength of reinforcement;
- Physical properties of epoxy adhesive;
- Depth and spacing of installed barrier reinforcement;

Needless to say that the barrier-deck interface is required to be made rough and clean. Roughening of surface between 6mm to 10mm maximum is specified. In addition, application of bonding agent prior to barrier concreting is recommended to ensure the interface has a minimum strength at least equal to the specified concrete strength.

It is worth noting that optimum spacing of reinforcement was dictated by the available depth of drilling. In many instances, only limited depth of penetration was available at tip of overhang. In case the final optimal arrangement including reinforcement spacing and available drilling/embement depth could not develop the ultimate pull out resistance of reinforcing bars, a reduction factor was applied based on the bonding adhesive properties as well.
Figure 7: Installation of new barrier on existing overhang

7.1.2 Bridge Strengthening

As per section 2.0 above, if the bridge capacity is deemed insufficient to resist applied loads (dead, live and collision) as per different limit states, then strengthening of bridge deck would be required. Several alternatives are thus investigated as presented in Figure 8.

1- Complete or partial demolition of the bridge overhang and reconstruction with additional top steel as deemed necessary; installed reinforcement will be adequately lap spliced with the existing overhang steel reinforcement as well as the new barrier reinforcement allowing for a monolithic barrier-deck connection. This option’s drawback is higher time and cost of construction and lengthy traffic interruption;

2- Thicken of overhang section from the bottom (soffit of slab) therefore inducing minimal obstruction on the bridge traffic. Some limitations are present as to road traffic below. From a structural perspective, even though the alternative allows for thickening of concrete section and larger fixation depth of new barrier reinforcement, it does not allow to increase tension reinforcement at top of overhang where it counts most;

3- Applying fiber reinforced polymer strips/laminates or rods to the top surface of the overhang. FRP strengthening has been recently more and more accepted as a reliable and efficient alternative for strengthening existing structures. It has the advantage to require only a minimal additional thickness (including protection) thus minimal additional loads and relatively high tensile strength to resist applied moments. For the case at hand, some of the drawbacks of the alternative include high cost of installation including surface preparation and installation, protection layer to mitigate traffic effect as well as temperature effect on the FRP laminates mainly during hot asphalt installation;

4- Concrete overlays at top surface of the overhang. A 10cm thick layer was deemed satisfactory. Concrete specifications are developed to ensure enhanced reinforcement protection and waterproofing, enhanced durability of bridge deck top surface and better resistance to wearing. The additional thickness provided allows for optimal connection between bridge deck and the new concrete barrier. This alternative also allows faster installation especially that no formwork is required for new concrete overlays and therefore minimum traffic interruptions which can be phased for right and left lanes independently. To mitigate the effect of additional dead loads on the bridge deck, concrete overlays layer will replace existing asphalt layer bearing in mind that the new
Concrete overlays will be fully bonded to the existing section and thus act as integral part of the final section leading to higher capacity of the same. Some additional advantages of the concrete overlays alternative include competitive unit rates and skills that are readily available in the local market, does not require specialist involvement as compared to FRP strengthening, implies upgrading of the bridge deck itself especially that majority of bridges in question are built between 1980 and 1985 and have their top surface material (concrete and reinforcement) in need of rehabilitation to extend their service life.

![Diagram of concrete overlays]

Figure 8: Various ways of strengthening an existing deck slab overhang

### 7.2 Construction Methods

Despite of the traffic detours at time of construction and its direct influence on level of service and road safety, the proposed sequence of construction for the scope of structural works for the concrete overlays at top surface bridge slab as explained in previous sections shall include the following steps:

1. Demolition/removal of existing railings/barriers consisting mainly of steel barriers or combination barriers (concrete-steel);
2. Removal of asphalt and wearing surface layers and other non-structural elements (curbs, pavement, screed, fill, etc...);
3. Preparation of bridge deck top surface to receive concrete overlays;
4. Installation of barrier dowel bars and barrier vertical and horizontal reinforcement;
5. Installation of bridge deck top mesh reinforcement;
6. Pouring concrete overlays layer and subsequent curing works;
7. Installation of new barrier formwork and pouring concrete as specified;

Special attention shall be given to infrastructure works and related connection points such as drainage pipe works, electrical conduits and installations, etc... Coordination between various trades and approval of authorities having jurisdiction shall always be solicited.

### 7.3 Traffic Detours at Construction Zones

Traffic detours at construction zones has a direct implication on the proposed solution due to its effect on both traffic requirement in terms of level of service and road safety. Thus, a reasonable proposed solution shall entail a comprehensive study on all aspects of traffic detours at construction zones in compliance with the codes and standards depicted by local authorities.

The two most critical aspects for traffic detours are:

- **Traffic Serviceability:** The current road traffic need to be uninterruptedly met throughout the construction period while providing a reasonable alternative route or ensuring that the temporary traffic detour serves the predicted demand
- **Road Safety:** Adequate signing and marking, buffer zones, temporary barriers and tapers shall be met to provide all safety maneuvers for both vehicular and pedestrian use.

### 7.4 Value Engineering

Several scenarios were investigated herein to identify relationships that increase the value of the proposed solution at a reasonable cost. The proposed alternatives are identified as follows:
- Alternative 1: the supply and installation of new concrete barriers for test level 5 by implanting additional dowels on the bridge slab for barrier fixation and reinforcing the bridge slab with a layer of reinforced concrete on the tile surface.
- Alternative 2: Supply and installation of new concrete barriers for test level 5 by implanting additional dowels on the bridge slab for barrier fixation and strengthening the bridge slab via polymer fibers.
- Alternative 3: Supply and installation of new concrete barriers for test level 4 by anchoring and reinforcing the bridge slab with a layer of reinforced concrete on the tile surface.
- Alternative 4: Supply and installation of new concrete barriers for test level 4 by anchoring and strengthening the bridge slab via polymer fibers.
- Alternative 5: Supply and installation of new concrete barriers for test level 4 without reinforcing the bridge slab.

It is obvious that the selection of the most valued alternative at a reasonable cost is dependent foremost on the proposed test level as well as to the capacity of the bridge itself. If more than one alternative is technically feasible, the complexity and cost of the alternative shall govern. A preliminary quantification and cost estimation of critical projects’ elements were identified and classified as follows:

- General site preparations including
  - Removal of Obstacles
  - Excavation works and removal of existing paving layers
  - Removal of existing barriers
- Traffic detours at construction zones including
- Strengthening of the bridge slab through overlays and reinforcement
- Pavement Works
- Supply and installation of polymer fibers to support the bridge slab
- Supply and installation of the new barriers
- Incidental works

Accordingly, the following points are taken into consideration in estimating the costs:
- The above estimates are based on the prevailing prices for similar construction items in KSA region, specifically Riyadh capital
- A 10% contingency has been added to cover unforeseen increases in prices and quantities, as the proposed solutions are estimated at a preliminary level.
- The width of the fiber-reinforced bridge slab was estimated at 3.5 m, including the required range of barrier fixation.
- Upon bridge strengthening, the area supported by the additional concrete layer is estimated at 10 meters wide from the concrete barrier (the bridge end) itself, and the work involves removing the paving layers and preparing the site and then restoring the asphalt surface layer
- The preliminary estimates did not show a significant difference in the cost of concrete barriers fixation either through dowel reinforcement or clamps on anchors. The pros of clamping the anchors for each barrier section are the speed of installation and its minimal impact on traffic. The method is mostly recommended if it happens to comply with both design forces and the structural integrity.
- The relative high cost of polymer fibers is due to the scarcity of such material and its complexity requiring specialized man power to complete the work.

Table 2 below summarizes the cost estimates for each alternative in linear meters:

<table>
<thead>
<tr>
<th>Alternatives</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Proposed Test Level</td>
<td>5</td>
<td>5</td>
<td>4</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>Barrier Fixation</td>
<td>Dowels Implantation</td>
<td>Dowels Implantation</td>
<td>Clamp through anchors</td>
<td>Clamp through anchors</td>
<td>Clamp through anchors</td>
</tr>
<tr>
<td>Bridge Strengthening</td>
<td>Reinforced Concrete Overlay</td>
<td>Fiber Reinforced Polymers</td>
<td>Reinforced Concrete Overlay</td>
<td>Fiber Reinforced Polymers</td>
<td>N/A</td>
</tr>
<tr>
<td>Cost per bridge linear meter (SAR)</td>
<td>4,150</td>
<td>5,370</td>
<td>3,700</td>
<td>4,380</td>
<td>1,220</td>
</tr>
</tbody>
</table>
8.0 Study Outcome
The study addresses most of the design stoppers that may be encountered throughout the analysis and proposes different ways to mitigate them. Thus, the main features for assessing the bridge barrier rehabilitation requirements were identified and explained while specifying all different design alternatives with the pros and cons of each. Although the framework for assessment of bridge barrier rehabilitation requirements on existing overpasses is standardized, the outcome of such framework is not straightforward but case specific. Even some solutions may be infeasible for the time being, but could be most prominent in the future as the construction techniques get more developed. In all cases, it is evident that the generalized framework described herein is deduced and verified on a total of twenty-three primary interchanges in Riyadh city. The same approach may be similarly conducted and integrated on other set of circumstances.

9.0 References
RETOFFITING URBAN STREETS: A REAL-WORLD APPLICATION
(TRACK: 4.3. “Safer Streets by Design”)

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ABSTRACT
The United Arab Emirates (UAE) has historically adopted a vehicle-oriented approach to its street design. This has been a common design approach not only in the UAE, but also in other Arab countries, particularly those part of the Gulf Cooperation Council (GCC). Unfortunately, the vehicle-oriented urban street design approach contributes to problems such as increased levels of traffic congestion and travel delay, decreased safety levels, poorer public health, and increased environmental impacts. In addition, it does not foster the use of alternate modes of transportation such as public transport, cycling, and/or walking. Fortunately, design policies began to change recently. The Emirate of Abu Dhabi, specifically, has reset its street design priorities from being vehicle-oriented to instead prioritizing other street users such as pedestrians, transit users, and cyclists. The Abu Dhabi Urban Planning Council (UPC) has prepared the Urban Street Design Manual (USDM) which explicitly provides guidelines on what the new norm is in regards to street design in Abu Dhabi. USDM explicitly sets out design specifications for street components such as traveled way, medians, and pedestrian realm based on land use and road capacity. This paper illustrates how existing, vehicle-focused streets can be retrofitted into more pedestrian-/cyclist-friendly streets through relatively simple, low-cost design modifications. The paper also provides a brief discussion on what the expected benefits of its proposed design may be.

Keywords: Urban Streets, Design, Vulnerable Road User Safety

1. PROBLEM STATEMENT
A steady increase in urbanization has raised serious mobility issues in major urban centers around the world, which have been aggravated by economic growth and vehicle-oriented urban planning policies. Economic growth has raised automobile ownership and ridership to record levels, whereas urban sprawling has contributed to automobile dependence (Cohen 2006, Kuangdi 2016, Frumkin et al. 2004). In turn, this all has led to vehicle-oriented transport planning strategies resulting in reduced travel options as alternative travel modes are stigmatized. As shown in Figure 1, this is a vicious cycle of automobile dependency which has supported the need for vehicle-oriented urban street design. Such a vehicle-oriented street design approach has its roots in an automobile-dependent culture which believes that automobiles are the solution for urban mobility. As a result, the believers of this ideology share the view that urban street design needs to be primarily tailored to accommodate those behind the steering wheel.

![Figure 1. Vicious cycle of automobile dependency](image-url)
Historically, a vehicle-oriented street design approach has also been adopted not only in the United Arab Emirates (UAE), but also in the Gulf Cooperation Council (GCC) countries in general. Figure 2 shows an inequitable, vehicle-oriented urban street design in Abu Dhabi, part of a street network located in a residential neighborhood. This design may be defined as inequitable since it offers motorists wide, well-paved travel lanes while it does not provide proper space for vulnerable road users such as pedestrians or cyclists. Motorists are given design priority while vulnerable road users are neglected, as no pedestrian or cyclist facilities are provided. The result is higher-speed, higher-capacity facilities making driving more favorable than any other transport mode including cycling or walking. Thus, in such a car-oriented street environment, cycling or walking are indeed unsafe. Secondly, there is too much empty right-of-way and unutilized land in between residential plots. These issues set the stage for a street environment that is very unattractive to road users other than motorists.

These design practices have proven to be unsustainable due to a number of reasons. First, research has shown that countries that have favored automobiles over transit, cycling, or walking have significantly higher levels of obesity (Bell et al. 2002). This is noteworthy because researchers have suggested that non-motorized transportation may help control body weight and prevent obesity (Frank et al. 2004, Calle et al. 1999). In addition, research has indicated that obesity has been linked to serious, chronic health problems such as hypertension, type II diabetes, cancer, heart disease, and stroke (Must et al. 1999, Field et al. 2001, Colditz et al. 1995, Burke et al. 2008, Drøyvold et al. 2005, Calle et al. 2003). Second, statistics show that over one million lives are lost due to traffic-related crashes annually (WHO 2015). Traffic injuries and deaths have indeed plagued the GCC countries which have suffered from high fatal traffic crash rates over the last several years (Al Khan 2013, Toumi 2010, Mannan 2017, Al Kuttab 2015, Kovessy 2014). Vulnerable road users have been found to be especially victimized, as they comprise a 30 percent share of the total number of traffic fatalities in the UAE even though vehicular volumes are significantly higher than pedestrian volumes in the entire country’s road network (Abdulla 2015 & Shahbandari 2017). This indicates an overrepresentation of pedestrians in the statistics related to traffic fatalities suggesting the presence of issues such as irresponsible road user behavior, inadequate traffic regulations, and/or poor road design. Lastly, motorized transportation contributes to degraded air quality which has been associated with an increased risk of respiratory diseases, cancer, brain disorders, and birth defects (Brunekreef & Holgate 2002, Cohen 1997, Ritz et al. 2002). The transportation sector is one of the major contributors to Green House Gases (GHG) worldwide. Almost 15 percent of the global GHG and over 20 percent of energy-related carbon dioxide (CO₂) emissions are produced by the transportation sector. Almost three-quarters of these emissions was derived from road transport (IPCC 2014).

Hence, given all these potential negative problems, there is a need to promote a more equitable urban street design in the GCC region.

2. OBJECTIVES

The objectives of this paper are two-fold: (1) to promote the urban street design principles contained in the Abu Dhabi Urban Planning Council’s (UPC) Urban Street Design Manual (USDM) (USDM 2010), as well as (2) to illustrate how existing, vehicle-focused streets can be retrofitted into more vulnerable-road-user-friendly streets through relatively simple, low-cost modifications.

3. ABU DHABI URBAN STREET DESIGN MANUAL

USDM was developed with the intent to equip designers with a guide which allows them to design more equitable urban streets accounting for the needs of not only motorists, but also of pedestrians, cyclists, and transit users as illustrated in Figure 3. This integrated approach is expected to address core areas such as: (1) land-use context which refers to the fact that varying land uses involve different activities along a street and, therefore,
their design must vary accordingly to accommodate those activities; (2) safety by making street design safer, especially for vulnerable road users; (3) efficiency through efficient operation of all travel models by increasing capacity in terms of number of people moved, increasing network connectivity, and decreasing travel distances and delay; (4) sustainability by adopting a less car-oriented street design which supports and encourages the use of other travel modes such as walking and cycling, decreasing vehicle emissions; (5) public health by designing more walkable streets which tend to increase physical activity levels and improve overall residents’ health; (6) public enjoyment which is a result of more livable, attractive streets; and (7) economic development by having the value of property and retail space increased as a result of better street design.

USDM is considered to be a departure from previous design guidelines adopted in the Emirate of Abu Dhabi such as the “Association of State Highway Transportation Officials’ Policy on Geometric Design of Highways and Streets” (AASHTO 2011). This US document may be appropriate for rural highways but is not as suitable for urban streets. USDM defines streets according to the typology presented in Table 1 which takes both land-use characteristics and transport-capacity capabilities into consideration.

USDM also provides guidance on sizing each design element of an urban street based on classifications presented in Table 1 as shown in Table 2. Basically, street cross-sections are divided into 3 main components (i.e., pedestrian realm, frontage lane, and traveled way) each being subdivided into a few zones. For example, pedestrian realm encompasses the frontage, through, furnishing, and edge zones, as well as the cycle track as shown in Figure 4. This is important because it helps design more complete streets by not only explicitly raising the need of segregating pedestrian facilities into parts, each part serving a purpose, but also by providing guidance on how to consistently size those parts according to their purpose. In the pedestrian realm, the frontage zone allows doors to be opened and passersby to window shop without disturbing pedestrian flow in the through zone. The furnishing zone serves to place street furniture in an orderly manner. The cycle track provides exclusive street space for cyclists, preventing potential conflicts between them and pedestrians or motorists. Finally, the edge zone provides a buffer space between bikes and vehicles.

<table>
<thead>
<tr>
<th>Street Family</th>
<th>Transport Capacity</th>
<th>Land Use Context</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Maximum Travel lanes</td>
<td>City (5+ Storeys)</td>
</tr>
<tr>
<td>Boulevard</td>
<td>3 + 3</td>
<td>City Boulevard</td>
</tr>
<tr>
<td>Avenue</td>
<td>2 + 2</td>
<td>City Avenue</td>
</tr>
<tr>
<td>Street</td>
<td>1 + 1</td>
<td>City Street</td>
</tr>
<tr>
<td>Access Lane</td>
<td>1 Shared</td>
<td>City Access Lane</td>
</tr>
</tbody>
</table>
4. RETROFIT DESIGN APPROACH

Preliminary retrofit design work prepared by Abu Dhabi Municipality’s (ADM) engineering team has been selected to illustrate how an older, vehicle-oriented design may be retrofitted according to USDM’s urban street design principles. This project involved major retrofit design work in the Baniyas area, one of the oldest and most traditional areas in the Emirate of Abu Dhabi. Figure 5 shows the area covered by this project which included 18 sectors, spanning an estimated area of almost 20 square-kilometers. However, all the design discussions contained in this paper will be limited to the yellow, highlighted area which covers sector EB1-01 as shown in Figure 5. All design practices selected for sector EB1-01 were applied to the remaining 17 sectors.

The retrofit design approach pursued during this work followed a 3-step process as described below.

![Figure 4. USDM's pedestrian realm (USDM 2010)](image-url)
4.1 SITE VISITS

First, site visits were conducted in order to assess Baniyas’s existing urban street infrastructure problems. These site visits were done in groups of 2 or 3 engineering team members with expertise in road safety auditing, traffic engineering, and transport planning. Also, the visits were carried out both during the day and night in order to investigate any traffic or safety issues that could emerge during different times of the day.

Pictures taken during the site visits were revised at ADM’s office. Design issues that needed to be rectified were identified during office technical discussions. Figure 6 shows a few photos taken in sector EB1-01.

4.2 PRELIMINARY DESIGN CRITERIA

In order for the engineering team to systematically identify design issues, a set of preliminary design criteria needed to be defined. Seven preliminary design criteria were adopted as shown in Table 3.

Table 3. Retrofit design criteria
4.2.1 Sidewalk

A walkable neighborhood is one of the key attributes of an equitable urban street network system. Thus, it is important to design pedestrian facilities that not only properly accommodate pedestrian flow by providing adequate sidewalk width and dropped kerbs for disabled users, but that also are connected through strategically-located crossings.

4.2.2 Cycling

Providing cycling facilities may not be as essential as providing walking facilities, but it may promote cycling for shorter-distance trips, as well as for recreational purposes. One needs to keep in mind, however, that in a retrofit design, there may not be enough right-of-way to accommodate cycling facilities. In this case, shared travel lanes may be used as long as they are properly marked with horizontal and vertical signs since motorists are to share lane space with cyclists. Yet, due to safety concerns, this sort of arrangement should be restricted to low-speed, low-traffic-volume streets often located in purely residential neighborhoods.

4.2.3 Traffic Safety

Traffic safety is one of the leading non-natural causes of death worldwide, especially among the young (WHO 2015, Hamadeh & Ali 2013). In the GCC region, a significant number of traffic-related deaths involve vulnerable road users. In the UAE, pedestrians have accounted for approximately 30 percent of all traffic-related fatalities (Abdulla 2015). Therefore, special importance was given to traffic-safety-related issues during the site visits. As a result, a number of potential traffic-safety-related problems were identified such as the improper provision or lack of a pedestrian realm, overly wide streets, no traffic calming measures, and no pedestrian crossing points.

4.2.4 Parking

One of the major problems observed during the site visits was the lack of formalized parking. It was observed that because existing travel lanes are too wide, vehicles were parked along those lanes next to the kerb line. Also, just as in the scenario shown in Figure 2, there was a significant amount of empty and unutilized right-of-way in sector EB1-01. This excessive amount of empty land had also allowed for the propagation of vehicles being randomly parked. This created not only an aesthetically unpleasant view as streets looked unorganized, but also an operational issue as these parked vehicles obstructed spaces meant to be used by pedestrians.

4.2.5 Mobility

Traffic congestion should always be considered in any urban transportation project. However, major mobility concerns were not found throughout the residential neighborhoods of Baniyas. Therefore, traffic congestion relief measures did not have to be implemented in this retrofit project.

4.2.6 Landscaping

Last but not least, landscaping is also relevant in order to make the street surroundings livable and inviting for residents to come out and explore. A bare street design as shown in Figure 2 may be unattractive, or even prohibitive, to walking or cycling activities. On the other hand, adding a few features such as vegetation, as well as a few shade-providing structures and benches along the sidewalks may produce a number of benefits including a more physically active neighborhood.

4.3 PRELIMINARY DESIGN OPTIONS

Once the existing issues had been identified via site visits and technical discussions, preliminary design options were investigated based on the six design criteria shown in Table 3. However, in order for a preliminary retrofit design to even be an option, it had to match the following prerequisites: (1) cross-section design must be accommodated within the existing available right-of-way, (2) minimal modification of the existing built infrastructure is required, and (3) proposed cross-section design must be a low-cost option.

It may seem that prerequisites (2) and (3) are redundant since less modification translates into lower costs. However, prerequisite (2) is concerned more with residents’ acceptance towards the proposed...
modifications. For instance, ADM could potentially face a lot of residents’ resistance if most of their well-taken-care-of gardens located in front of their villas had to go away. The purpose of prerequisite number (2), therefore, was to minimize major modifications that could potentially trigger residents’ dissatisfaction and resistance.

Within each sector, streets were then classified into 3 main classes as shown in Figure 7: (1) internal streets, (2) external streets, and (3) avenues. Internal streets are those that are located inside a sector’s boundaries. External streets are those that delineate a sector. Even though avenues are located similarly to external streets, they are multilane, divided roads.

At least three preliminary design options were prepared for each street class, and the best candidate was selected based on the three prerequisites listed previously.

![Figure 7. Street classes in sector EB1-01](image)

4.3.1 INTERNAL STREETS

The total available right-of-way for the internal street shown in Figure 7 is 32 meters. Therefore, according to prerequisite (1) discussed in section 4.3, the retrofitted cross-section containing all design improvements based on criteria discussed in section 4.2 should be accommodated within 32 meters. Figure 8 shows the proposed retrofitted plan and cross-section views of the selected preliminary design option for this internal street.

![Figure 8. Plan and cross-section views of internal street in sector EB1-01](image)

Starting from the middle of the road towards the residential plot boundaries, the modifications made are as follows. First, the travel lane width was reduced from over 4 to 3 meters. This reduction in travel lane width presents at least 3 benefits: (1) other spaces, such as pedestrian realm, may be created or expanded while still keeping the right-of-way width unchanged; (2) narrower road width may have a traffic calming effect by
lowering operating vehicle speeds, making the street environment safer, especially for vulnerable road users; and (3) less road exposure to pedestrians, as they need to cross shorter distances to cross travel lanes.

Second, continuous 2-meter wide sidewalks have been proposed. It is important to note that sidewalks present in the existing street infrastructure are narrower and have utility poles placed right in the middle, leaving little room for pedestrian flow, especially for those utilizing strollers or wheelchairs. There are also instances where no sidewalk has been provided. In addition, because travel lanes were significantly narrowed, parallel parking bays were inserted in the surplus space created. This way, most of the new, continuous and wider proposed sidewalk stays where the existing sidewalk is. Yet, in order to become a 2-meter wide sidewalk, the existing sidewalk will have to be expanded laterally (i.e., towards the plot boundaries). By doing this, the existing utility poles that currently sit in the middle of the existing sidewalk will be located on the inner edge (i.e., edge close to the kerb line) of the proposed sidewalk, strategically located either in between the two parking bays or in the landscape area as represented by the stars shown in Figure 8. In order to place the poles as close to those stars as possible, poles will need to be moved longitudinally (i.e., along the street), but not laterally. This shall avoid enormous costs associated with relocation of underground utilities which is in line with prerequisite (3) described in section 4.3 (i.e., proposed cross-section design must be a low-cost option). It is also true that moving light poles along the street may affect lighting itself. Thus, after discussing the issue with ADM’s lighting engineers, the engineering design team concluded that moving the poles by no more than 2 meters longitudinally from their original location would not significantly affect lighting coverage.

Third, in order to accommodate cyclists, horizontal signs marked on the travel lanes, as shown in Figure 8, warn road users that road space is to be shared between automobiles and cyclists. While it would be ideal to provide exclusive lanes for cyclists, right-of-way width restrictions did not allow it. Nevertheless, these internal streets carry little traffic and vehicles are expected to travel at average operating speeds of no more than 30-kph once the proposed traffic calming measures are implemented. Thus, serious safety issues associated with potential conflicts between automobiles and cyclists are expected to be mitigated.

Fourth, traffic calming measures such as speed humps, speed tables, and raised junctions have been proposed as a means to keep operating vehicle speeds in the 30-40-kph range. Past traffic calming studies have proposed speed humps to be spaced 89-152-meters apart in order to keep speeds within the 40-48-kph range (Weber 1998, Parkhill et al. 2007, Robinson et al. 1997, Hallmark et al. 2002). In the internal street design presented in this paper, traffic calming measures have been proposed to be spaced 100-130-meters apart.

Fifth, parking bays were marked on each side of the street, as well as on each plot’s driveways, totaling 8 parking spaces for each villa. This will better delineate pedestrian and vehicle spaces, creating a more organized street layout.

Lastly, a significant amount of space dedicated for landscaping was kept. In the existing street scenario, many of the residents have gardens planted in front of their villas; therefore, the engineering team considered it important to keep as much of those areas intact as possible. Even though these areas may become smaller with the proposal of six parking spaces in each villa’s driveways, there will still be a significant amount of space available, approximately just over 30 square-meters, in front of each villa. This is not counting the area next to the sidewalk and on-street, parallel parking bays.

4.3.2 EXTERNAL STEETS

Figure 9 shows the plan and cross-section views of the selected preliminary design option for the external street located in sector EB1-01. The total proposed right-of-way to be used for this street is 20 meters, though the total available right-of-way on this street is wider than 20 meters.
Figure 9. Plan and cross-section views of external street in sector EB1-01

As shown in Figure 6, there are a number of problems along this street such as very wide travel lanes, limited sidewalk space due to narrow sidewalk width and misplaced light poles, no cycling lanes, no landscaping, overgrown vegetation, very spaced out traffic calming measures, and no parking provision.

In order to keep costs to a minimum, the existing kerb line remained unchanged. The existing travel lanes are wide enough to accommodate the proposed narrower lanes plus the proposed parallel parking spaces as shown in Figure 9. The pedestrian realm is to be expanded by creating a furnishing zone and a 2-meter wide, unobstructed sidewalk on the opposite side of the internal street accesses. The furnishing zone was created to accommodate street furniture such as light poles, benches, and shade-providing structures. On the opposite side of the internal street accesses, there are a number of schools and, therefore, plenty of parking spaces were provided, as well as speed tables for traffic calming and safer pedestrian crossing purposes. Lastly, a 3-meter wide, two-way cycling route has been proposed on this same side of the street where cyclists do not have to cross junctions, decreasing the potential for bike-car conflicts.

4.3.3 AVENUES

Figure 10 shows the plan and cross-section views, respectively, of the selected preliminary design option for the avenue street in sector EB1-01. The total proposed right-of-way to be used for this street is 35.8 meters, though the total available right-of-way on this street is wider than this. Again, in order to keep costs to a minimum, the existing kerb line may remain unchanged.

This is a divided, two-lane, higher-speed-limit street with posted speed limits of 60-kph as opposed to 40-kph on the other street classes. However, as per traffic regulations in the UAE, motorists are legally allowed to drive up to 20-kph above the posted speed limit before they are fined. Therefore, ordinary traffic calming measures such as speed humps and tables have not been proposed on this street class.

The retrofit design proposes to replace the narrow and obstructed existing sidewalk with an edge zone which can serve as a buffer between high-speed moving vehicles and cyclists. A 3-meter wide, two-way cycling route has been proposed. Next to it, a 1-meter wide furnishing zone and a 2.5-meter wide sidewalk have been proposed as shown in Figure 10.
5. SUMMARY

Problems associated with the prevalence of an outdated, vehicle-oriented urban street design approach in the GCC region have been raised. As an acknowledgement that this design approach may not be optimal, the Emirate of Abu Dhabi, through its UPC, developed USDM. The urban street design guidelines contained in USDM match well with international best practices (NACTO 2013), and they tailor design such that vulnerable road users such as pedestrians, cyclists, and transit users take priority over automobiles. The principles contained in USDM are presented in this paper, and their application is illustrated through a real-world application with a number of retrofitted, preliminary street designs being proposed for the Baniyas area in Abu Dhabi, UAE.

In order to retrofit these sectors, site visits were conducted in order to identify existing issues based on the six design criteria described in section 4.2. Streets within each sector were classified into three different classes: internal streets, external streets, and avenues. Potential design options were investigated for each street class based on pre-established design prerequisites described in section 4.3. Finally, the final selected preliminary retrofitted designs were presented.

6. BENEFITS

The existing street infrastructure in Baniyas presents a number of issues. These issues may be mainly linked to the lack of focus on vulnerable road users. Shifting the design focus towards these users is crucial for residents to potentially reap benefits such as increased physical activity levels and improved traffic safety.

These are two very important aspects to deal with throughout the GCC region. Statistics show that GCC countries’ populations are among the most overweight in the world (Saberi 2012, Swanson 2015, Al Khan 2014). While GCC countries have become relatively affluent nations with some of the highest income per capita in the world (Gregson 2017), poor diet coupled with low levels of physical activity have plagued GCC residents with preventable health problems such as obesity, heart disease, stroke, and diabetes (Saberi 2012, Chaudhary 2014, Strategy& 2014). The good news is that, as shown in Figure 11, past research has suggested that countries which have high percentages (i.e., as high as 50 percent) of walkers, cyclists, and transit users such as the Netherlands, Switzerland, and Denmark have significantly lower obesity levels as compared to countries which have very low percentages (i.e., as low as 5 percent) such as the United States (Bassett et al. 2008). However, proper road infrastructure is needed in order to promote walking and cycling. If roads are pedestrian hostile, then walking or cycling could entail more risk than benefit due to undesirable safety levels.
Traffic safety is another benefit to be considered. Several streets with improper sidewalks, or none at all, have been spotted in Baniyas’s existing infrastructure. In addition, because no formal parking spaces have been provided, vehicles park randomly blocking even the limited pedestrian facilities currently present. Also, because roads are currently too wide and no limited traffic calming measures have been adopted, ADM has received innumerable resident complaints concerning motorists driving too fast around their neighborhood, placing pedestrians at risk, especially children. However, by providing a clear separation between vehicles and vulnerable road users, as well as by narrowing travel lanes and implementing traffic calming measures, the proposed retrofit design anticipates improved traffic safety. These are important considerations, as a large percentage of traffic fatalities in the GCC region are pedestrian related (Abdulla 2015, Mannan 2017).

7. LIMITATIONS

It is important to reinforce that the design work presented in this paper is a retrofit design. As such, it is not necessarily the best possible option for those sectors in Baniyas. However, as mentioned in section 4.3, design options had to meet three prerequisites in order to be selected: (1) cross-section design must be accommodated within the existing available right-of-way, (2) minimal modification of the existing built infrastructure is required, and (3) proposed cross-section design must be a low-cost option. Hence, expanding the right-of-way in the name of designing more complete streets was not an option. This would also mean interfering too much with outdoor areas currently being used by residents, as well as increasing costs exponentially.

It is also important to stress that there was no formal resident engagement during the design work presented in this paper. There was also no resident opinion survey conducted. As a result, it is not possible to know the degree to which residents would be in favor of the proposed design. Nevertheless, designers are strongly encouraged to consult and engage residents during their design work. By engaging residents and incorporating their input into the design process, residents become aware of existing problems, as well as educated on why changes need to take place.

It is also important to stress that there is no continuity between proposed cycling lanes from Baniyas outward because there currently are no cycling facilities in neighboring areas. However, in order to promote major travel mode shifts from motorized to non-motorized transportation, not only does proper, citywide street infrastructure need to be provided, but cities also need to be planned accordingly. That is, cycling lanes need to have continuity and be connected, while urban planning needs to be done in such a way that people are less dependent on cars by shortening travel distances. This means planning cities more based on a traditional neighborhood concept rather than on a suburban sprawl one (Speck 2013, Cohen 2006).

Lastly, it is worth acknowledging that major travel mode shifts may also be dependent on cultural aspects which often only change slowly over time. That being said, one may argue that there is no need to expand provision of walking or cycling facilities in the GCC because this is a region that does not have a strong cycling culture, for example. However, one may also intelligently point out that there will not be a significant number of cyclists before proper cycling infrastructure is provided.

8. ACKNOWLEDGEMENTS

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9. REFERENCES


ABSTRACT:
According to the Wyoming Strategic Highway Safety Plan, lane departure crashes contribute to 72% of all critical crashes in Wyoming. Critical crashes are defined as events that lead to a fatality or incapacitating injury. In the US, single vehicle run off the road crashes contribute to a total societal loss of $80 billion every year. Shoulder Rumble Strips (SRS) have been proven to have a significant effect on reducing lane departure crashes. While it is simple to quantify the safety effectiveness of permanent shoulder rumble strips, the process becomes challenging when resurfacing and shoulder rehabilitation may result in an intermittent presence of SRS in some locations.
This paper quantified the safety effectiveness of shoulder rumble strips for three different conditions; 1) assuming that SRS are present at all times since their initial implementation, 2) considering only verified sections with existing SRS for a continuous period of time, and 3) considering the intermittent nonexistent time periods of SRS as before period with after period when SRS are implemented. The paper utilized cross-sectional and observational before-after with Empirical Bayes (EB) methods for the aforementioned three different conditions. Two sets of Wyoming-specific full Safety Performance Functions (SPFs) were developed for the cross-sectional analysis and the before-after with Empirical Bayes (EB). The results from this study indicate that the safety effectiveness of shoulder rumble strips might be underestimated in case of not considering the intermittent implementation practice in the state. Specifically, it was found that SRS are 35% and 20% more effective in reducing fatal and injury, and lane departure crashes when considering the intermittent implementation compared to assuming their continues presence, respectively. It is recommended that transportation agencies should consider immediate reapplication of SRS after resurfacing projects in order to help reducing fatal and injury, and lane departure crashes.
1. INTRODUCTION

According to the Fatality Analysis Reporting System (FARS), in the US, nearly 6.3 million motor vehicle traffic crashes were reported in 2015 (NHTSA 2015). The economic toll of all traffic crashes sums up nearly to $240 billion annually (Choi 2010). Of all crashes, 33,000 and 1.7 million crashes are fatal and injury, respectively. Researchers have identified roadway departure crashes as a leading cause of traffic fatalities and injuries on highways and also as a threat to traffic safety. According to the Federal Highway Administration (FHWA), from 2013 to 2015 an average of 18,275 fatalities resulted from roadway departures, which is 54 percent of all traffic fatalities in the US (FHWA 2017a).

A roadway departure crash is defined by the FHWA as “a crash in which a vehicle crosses an edge line, centerline or otherwise leaves the traveled way” (Jalayer & Huaguo 2016). From 2007 to 2013, fourteen states in the US have experienced more than 65 percent of all roadway fatalities because of roadway departure crashes. Vermont and Wyoming are the top two states with the highest percentage of roadway departure fatal crashes counting 79 and 78 percent, respectively.

Roadway departure crashes result in more severe run off the road (ROR) crashes or opposite head-on crashes. Different contributing factors for run off the road crashes have been identified in the literature. A study in Minnesota found that relative risk of run off the road fatalities increases as speed of the vehicle increases (Davis et al. 2006). Another study in Washington D.C. indicated that 25 percent of the driver decision errors because of speeding result in run off the road crashes (Liu & Ye 2011). McGinnis et al. (2001) utilized the FARS data to analyze fatal run off the road crashes from 1975 to 1997. The results indicated that alcohol plays an important role in roadway departure crashes (McGinnis et al. 2001). Dissanayake (2003) studied run off the road crashes using data from Florida in years 1997 and 1998. He identified speeding, alcohol, and gender as the significant contributing factors to ROR crashes (Dissanayake 2003). Another important factor responsible for run off the road crashes is driver’s drowsiness (Horne & Reyner 1995, Kozak et al. 2006, Royal 2002). The National Highway Traffic Safety Administration (NHTSA) data indicate that in 2002, there have been about 100,000 crashes because of driver drowsiness resulting in 1,500 run off the road fatalities which contributed to 8 percent of total run off the road fatalities (Kozak et al. 2006). According to the most recent Wyoming Strategic Highway Safety Plan (2012), a total of 1,532 lane departure/run off the road crashes resulted in fatality and/or incapacitating injury in Wyoming, from 2008 to 2010. This number of crashes had 72 percent share of all critical crashes in Wyoming (WYDOT 2012). These crashes were consequences of driver fatigue, impaired driving, speeding and distracted driving. Run off the road crashes occur more than twice as many as in rural roadways than in urban (Lord et al. 2011). Numerically, about 80 percent of the run off the road fatalities occur in rural roadways, with about 90% of those rural crashes occurring on two-lane highways (Lord et al. 2011).

One of the possible countermeasures to address roadway departure and its associated crashes, Shoulder Rumble Strips (SRSs) have been used in most of the states in the US. The safety of two-way two-lane highways extremely depends on drivers’ lane keeping ability. Nevertheless, non-intentionally drivers fail to maintain their lane for the aforementioned reasons, and hence possibility of being involved in a crash increases. Shoulder Rumble Strips (SRS) are defined as a series of raised or grooved strips or stripes along the sides of a roadway.

The main goal of the shoulder rumble strips is to produce vibrotactile and audible warnings to inattentive drivers departing their lanes (Harwood 1993). Shoulder rumble strips were first implemented in 1955 on 25 miles of New Jersey’s Garden State Parkway (Garder and Alexander 1995). Due to increasing documentation of the safety benefits of shoulder rumble strips, at present, more than 85 percent of the states use shoulder rumble strips in their roads (FHWA 2017b). The Wyoming Department of Transportation (WYDOT) started a SRS statewide implementation program starting 2002.

Several studies evaluated the safety efficacy of SRS in the US. Shoulder rumble strips are installed primarily to reduce lane departure and single vehicle run off the road (SVROR) crashes. The effectiveness of SRS to reduce and lane departure and SVROR crashes varied in the literature. Shoulder rumble strips were found to reduce 36 percent of SVROR crashes on average as reported in the NCHRP 641 report (Torbic et al. 2009). Kansas, Washington, Massachusetts, and Pennsylvania reported 3, 18, 42, and 60 percent reduction in SVROR crashes, respectively. A recent study in 2013 from Washington State Department of Transportation (WSDOT) was carried out to quantify the effect of shoulder rumble strips only and to estimate the effect of shoulder rumble strips and centerline rumble strips combined. The results indicated a 12.3 percent increase in lane departure crashes for all severity levels. It is worth mentioning that the study utilized a limited dataset on a short roadway segment of about 30.6 kilometers (19 miles) with shoulder rumble strips only and a total of 124.6 kilometers (77.44 miles) of roadway segments were selected as a combination of shoulder and centerline rumble strips. Overall, the study concluded 66 and 56 percent reduction in lane departure total crashes, and fatal and injury crashes, respectively (Olson et al. 2013). Shoulder rumble strips were also found to be effective in reducing total crashes occasionally, in addition to reducing fatal and injury crashes. A study in California concluded that 90 percent and 42 percent reduction in fatal, and total head-on crashes after
installing SRS, respectively (Karkle et al. 2012). The study by Olson et al. (2013) reported that the combination of shoulder and centerline rumble strips indicated about 63 and 43 percent reduction in total, and fatal and injury crashes, respectively. The study utilized naive before-after method using crash data from 2002 to 2010 in Washington. Different states have different CMFs for SRS ranging between 7 percent to 41 percent reduction in total crashes because of different traffic and geometric conditions (FHWA 2003). However, some other studies in Minnesota, Montana and Pennsylvania reported no reduction in total crashes after the installation of shoulder rumble strips. Another study by Park et al. (2015) concluded that the safety effectiveness increases with shoulder rumble strips in a 2.9 meters (9.5 feet) or above shoulder width (Park & Abdel-aty 2015). In this study, 381 roadway segments on a total of 437.7 kilometers (272 miles) roadway with shoulder rumble strips in Florida were considered for analysis. The study utilized before-after with Empirical Bayes (EB) method to carry out the analysis. The authors concluded that the safety effectiveness increases with the increase of shoulder width on roadway segments with shoulder rumble strips. A previous study in Wyoming showed that implementation of shoulder rumble strips can have an effect from no reduction to a reduction of total and F+I crashes by 1 to 29 percent using Weighted Severity Index (WSI), based on simple before-after crash frequency only (Coulter & Ksaibati 2013).

All of the aforementioned studies considered continuous presence of shoulder rumble strips, for which, it is simple to quantify the safety effectiveness of shoulder rumble strips. The impact of removing or intermittent application of a countermeasure has been investigated in the literature for other countermeasures such as red light running cameras (RLRCs). Red light running cameras at intersections are used in a number of the US cities as a countermeasure to reduce red light running violations and their associated crashes. Florida used red light running cameras in Orange County, Orlando and Apopka on a rotation basis. Red light running cameras are used at specific intersections for a specific time period and then rotated to other intersections. A study on the efficacy of RLRCs by Ahmed and Abdel-Aty (2014) in Florida found a reduction in red light running violations and their associated crashes not only at intersections equipped with RLRCs but also a spillover effect on the nearby intersections which were not equipped with cameras (Ahmed & Abdel-aty 2015). Retting et al. (1999) also concluded a change in drivers’ behavior in California and Virginia because of red light running cameras (Retting et al. 1999a & Retting et al. 1999b). However, the effect of intermittent application of shoulder rumble strips may not have the same effect. Red light running cameras is a behavioral countermeasure that meant to change driver’s behavior. On the other hand, shoulder rumble strips is a physical location-specific countermeasure. Implementation of shoulder rumble strips alerts a driver at the moment of departing a travel lane. An intermittent application of shoulder rumble strips makes the process of quantifying the safety effectiveness of shoulder rumble strips challenging. With the intermittent use, the shoulder rumble strips are taken off along with the implementation of an overlay or resurfacing treatment. Because of the absence of shoulder rumble strips, lane departure crashes may rise and the exact safety effects of shoulder rumble strips may not be correctly reflected in the safety effectiveness evaluation. There has been no studies that looked into the impact of project management strategies, i.e., the intermittent application of shoulder rumble strips in this study, and hence, quantify the safety effectiveness of shoulder rumble strips is crucial. According to the Wyoming Department of Transportation (WYDOT), shoulder rumble strips are removed whenever an overlay or a resurfacing project takes place. Shoulder rumble strips will be reinstalled after 2 years when there are enough lengths for a districtwide implementation. However, this could be a rough estimation that might not be applicable for all cases. Therefore, the main objective of this study is to assess the impact of the intermittent existence of SRS on the safety of two-way two-lane highways in Wyoming. The concept of analysis used in this study will be helpful for transportation agencies to evaluate and justify safety improvements because of the intermittent implementation of other countermeasures, e.g., centerline rumble strips.

To achieve this goal, three different conditions for shoulder rumble strips were considered; 1) base condition by ignoring the intermittent presence of shoulder rumble strips, 2) considering only the sections that have shoulder rumble strips for a continuous period of time, and 3) considering the intermittent nonexistent time period of shoulder rumble strips as before period with the after period as having the shoulder rumble strips at the same segments.

The remainder of this paper consists of data preparation and description, statistical methods used for analysis, detailed results of the safety efficacy of intermittent implementation of shoulder rumble strips using three different conditions with discussions followed by conclusions and recommendations.

2. DATA DESCRIPTION AND PREPARATION

The Highway Safety Manual (HSM) provides Safety Performance Functions (SPFs) for rural two-lane highways to predict crashes. However, the SPFs provided in the HSM may not be applicable to Wyoming-specific conditions. Therefore, Wyoming-specific full SPFs were developed in this study for rural two-way two-lane highways to predict crashes. This study also utilized two observational analysis methods; Cross-sectional, and before-after with Empirical Bayes (EB). The data needs are quite extensive to perform these statistical analyses. Multiple data sources
were used to meet the data requirements. The main dataset used in this study was the historical crash data in Wyoming. WYDOT records and digitizes crashes occurred on Wyoming’s road network. Raw crash data could be accessed via the Critical Analysis Reporting Environment (CARE) software. The crash data were separated into three categories; 1) Total crashes, 2) Fatal and Injury (F+I) crashes, and 3) Lane Departure crashes (LDC). Traffic data including annual average daily traffic (AADT), truck percentages, implementation dates of countermeasures, and roadway characteristics such as vertical and horizontal road geometry, were obtained from WYDOT. However, several limitations were encountered in the data collection step. Non-traditional data sources were utilized to overcome these data gaps. It is worth mentioning that implementation dates for countermeasures in different sections are one of the most crucial information needed to conduct an observational before-after analysis and was the most challenging task. Missing implementation dates were imputed by an extensive manual data collection effort utilizing Pathway Video Logs as well as by navigating through the Google Earth Pro® and Google Map Street Views. In addition, weather is considered a critical component contributing to crashes in Wyoming. Weather stations information obtained from the National Oceanic and Atmospheric Administration (NOAA) were utilized for the study locations.

Variables used to develop the SPFs were categorized into four groups: crash data, geometric characteristics, traffic data, and weather data. Types and levels for each variable belongs to these categories are shown in Table 1.

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<th>Max.</th>
<th>Min.</th>
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<td>VG2-164.2 km (102 miles)</td>
<td>VG3-193.1 km (120 miles)</td>
<td>VG4-48.3 km (30 miles)</td>
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<td>Natural Log of Vehicles Traveled</td>
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<td>Natural logarithm of product of AADT and length of segment; ln of vpd-km (vpd-miles)</td>
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<td>Total crashes (crash/year)</td>
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</table>

The data were collected from rural two-way two-lane highways in Wyoming. The initial data consists of a total of 476 kilometers (296 miles). The examined roadways were divided into 709 segments. The data were collected for 12 years from 2003 to 2014. Homogeneous segmentation method was followed in segmentation of the investigated roadways using the vertical and horizontal geometric characteristics. A total of 656 fatal and injury crashes were observed out of 2460 total crashes over these years.
As mentioned earlier, Pathway Video Logs were used to overcome gaps in the data such as gaps in implementation year of a countermeasure. However, some issues were also identified while navigating through these video logs. Shoulder rumble strips (SRS) were widely implemented in the State of Wyoming starting in 2002. While verifying the implementation status of SRS in Wyoming, Pathway Video Logs indicated that SRS might be removed because of an overlay implementation. The video logs indicated that SRS were reinstated after several years for these locations. This intermittent presence of SRS is due to the cost-effective project management strategies in Wyoming. Several roadway segments with new overlay application should be combined to allow for a wide jurisdiction reimplementation of SRS. Observational before-after studies assume a consistent presence of countermeasures; once a location receives a certain treatment, it is assumed that it always exists in the after period. Figure 1 shows an example at US 20/26 at kilometer post 67.87 (milepost 42.173) where SRS existed in 2012 and were absent in 2014 when a new overlay was performed in 2013. Lane departure crashes also increased when shoulder rumble strips were removed. There were 5 lane departure crashes per year before the overlay was applied when there were shoulder rumble strips. But soon after the application of new overlay, the shoulder rumble strips were removed and there were 8 lane departure crashes per year. It is worth mentioning that the AADT in that roadway segment did not change between 2010 and 2015. This may result in biased estimates of CMFs when assuming a consistent presence of SRS after the initial implementation date. Similar conditions were experienced by some other roadway segments like US 20/26 (kilometer post 109.4-127.9/milepost 68-79.5), US 85 (kilometer post 101.4-117.5/milepost 63-73), US 26 (kilometer post 15.8-23/milepost 9.8-14.3), etc. Table 2 describes lane departure crash rates in before and after years of an overlay for these roadways.

![Image 1](Original Photo © 2016 Pathway Video Logs)

**Figure 1: Shoulder Rumble Strips Intermittent Presence**

<table>
<thead>
<tr>
<th>Route</th>
<th>Length in km (miles)</th>
<th>Lane departure crash rate per 100 MVKT (MVMT) (Before an overlay, with SRS)</th>
<th>Lane departure crash rate per 100 MVKT (MVMT) (After an overlay, without SRS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>US 20/26</td>
<td>33.3 (20.7)</td>
<td>20.32 (32.70)</td>
<td>25.22 (40.57)</td>
</tr>
<tr>
<td>US 20/26</td>
<td>18.5 (11.5)</td>
<td>14.54 (23.40)</td>
<td>19.70 (31.69)</td>
</tr>
<tr>
<td>US 85</td>
<td>16.1 (10.0)</td>
<td>29.54 (47.56)</td>
<td>40.72 (65.56)</td>
</tr>
<tr>
<td>US 26</td>
<td>7.2 (4.5)</td>
<td>37.14 (59.43)</td>
<td>43.10 (68.96)</td>
</tr>
<tr>
<td>Total</td>
<td>75.1 (46.7)</td>
<td>21.64 (34.80)</td>
<td>27.57 (44.34)</td>
</tr>
</tbody>
</table>

Table 2: Lane departure crash rate before and after of an overlay
The intermittent application of shoulder rumble strips was not considered in the initial analysis and a continuous presence of shoulder rumble strips was assumed after their initial implementation. However, after identifying the intermittent application issue of SRS through Pathway Video Logs, the data had to be verified for each year in terms of the existence of SRS. Investigated number of years were reduced to 7 years starting from 2008 to 2014 instead of 12 years from 2003 to 2014 by utilizing Pathway Video Logs and a list of projects accomplished by the Wyoming Department of Transportation. The Pathway Video Logs were available from the year 2010 to 2015. The implementation status of shoulder rumble strips was identified and extended to the year 2008 by comparing the video logs to project list accomplished by WYDOT between 1996 and 2014. WYDOT collects video data for a roadway segment every other year. The video logs for most of the two-way two-lane highways were captured in 2014. Also, the list of completed projects by the Wyoming Department of Transportation was available up to the year 2014. That is why the dataset could not be extended beyond 2014.

A total of 280 roadway kilometers (174 miles) were selected as reference sites (sites that did not receive any treatment, such as shoulder rumble strips) to develop final SPFs. Again, homogeneous segmentation method was followed in segmentation of the investigated roadways of the final dataset. The examined reference sites were divided into 514 segments. The average AADT was 1,529 vehicle per day (vpd) and average truck percentage of 14.5 percent which is lower than the original dataset. A total of 854 crashes were recorded from 2008 to 2014 time period. Out of 854 crashes, 148 were fatal and injury crashes and 543 were lane departure crashes which indicated 17 percent of total crashes were fatal and injury crashes and 64 percent of all crashes were lane departure crashes.

To overcome the intermittent application issue of SRS for treatment sites (sites with shoulder rumble strips), two approaches were adopted. First, the particular sections that received intermittent application of SRS, were dropped from analysis and only sections with continuous presence were selected. Another approach was considering every off situation as before period and every on situation as an after period which will be defined as before-after for individual segments. A total length of 75.3 kilometers (46.8 miles) were selected as treatment sites from US 20/26, US 85 and US 212 as continuous implementation of shoulder rumble strips. Another 75.2 kilometers (46.7 miles) were selected as treatment sites from US 20/26 and US 85 which experienced intermittent use between 2007 and 2014 and were combined with the continuous implemented treatment sites as another approach of analysis. The summary of data inventory is provided in Table 3.

### Table 3: Summary of Data Inventory

<table>
<thead>
<tr>
<th>Condition</th>
<th>Treatment Sites</th>
<th>Methods Used</th>
<th>Years</th>
<th>Total Length in kilometers (miles)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Assuming Continuous Presence of Shoulder Rumble Strips</td>
<td>All Treatment Sites</td>
<td>Cross-sectional</td>
<td>2003-2014</td>
<td>476 (296) (Total)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Before-After EB</td>
<td>1997-2001 (Before)</td>
<td>154 (96) (Treatment Sites)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2004-2008 (After)</td>
<td>322 (200) (Reference Sites)</td>
</tr>
<tr>
<td>Considering Intermittent Implementation Only of Shoulder Rumble Strips</td>
<td>Treatment Sites with Continuous Implementation Only</td>
<td>Cross-sectional</td>
<td>2008-2014</td>
<td>355 (220.8) (Total)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Before-After EB</td>
<td>2007-2009 (Before)</td>
<td>75.3 (46.8) (Treatment Sites)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2012-2014 (After)</td>
<td>280 (174) (Reference Sites)</td>
</tr>
<tr>
<td></td>
<td>Before-After for Individual Segments (considering on/off SRS implementation)</td>
<td>Before-After EB</td>
<td>2007-2014</td>
<td>150.5 (93.5) (Treatment Sites)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>280 (174) (Reference Sites)</td>
</tr>
</tbody>
</table>

### 3. METHODOLOGY

Safety Performance Functions (SPFs) are mathematical models used to predict average crash frequencies per year as a function of exposure and roadway characteristics. The predictive methods in the HSM offer Safety Performance Functions (SPFs) as a quantitative measure for estimating yearly expected crash frequencies for base conditions. The base conditions for roadway segments on rural two-lane two-way roads are provided in the HSM (AASHTO 2010). However, the condition for a specific site might not be same as the sites used to develop Safety Performance Functions provided in HSM. Therefore, Wyoming-specific full SPFs for rural two-way two lane
highways were developed in this study using various prediction models. Five different models were applied to develop the crash prediction models: 1) Poisson Model, 2) Negative Binomial Model, 3) Zero Inflated Poisson Model, 4) Zero Inflated Negative Binomial Model and 5) Log Normal Model. The best fit model was selected based on the Akaike Information Criterion (AIC). In addition to these models, two statistical observational analysis methods were applied to quantify the safety effectiveness of shoulder rumble strips: 1) Cross-sectional analysis and 2) Before-after with Empirical Bayes (EB).

3.1 Negative Binomial Model (NB)

Negative Binomial distributions describe the occurrence of random and rare events. However, unlike the Poisson distribution, where it is assumed that the mean is equal to the variance, the Negative Binomial distribution compensates for situations where the variance is greater than the mean, or when the data is overdispersed. The Negative Binomial (Poisson-Gamma) model is obtained by the assumption given in Equation (1).

\[ e^{(\mu_i, \kappa)} \sim \text{Gamma}(\kappa, \kappa) \]  

where \( \kappa \) is the inverse dispersion parameter. The dispersion (or over-dispersion) parameter is usually referred to as \( \beta = 1/\kappa \). The probability density function of the NB model is given by Equation (2) (El-Basyouny 2011).

\[ \Pr(Y_i = y_i | \mu_i, \kappa) = \frac{\Gamma(y_i + \kappa)}{\Gamma(y_i) \Gamma(\kappa)} (\frac{\mu_i}{\kappa + \mu_i})^{y_i} (\frac{\kappa + \mu_i}{\kappa})\kappa \]  

Under the NB model, the mean and variance are given by Equation (3).

\[ E(Y_i) = \mu_i, \quad \text{Var}(Y_i) = \mu_i + \mu_i^2/\kappa \]  

When mean will be equal to variance, \( \beta \) will go to zero and NB model would be transformed into a Poisson model. The Negative Binomial regression model has been widely applied in the road safety literature.

3.2 Log-Normal Regression Model

Negative binomial model addresses the discrete response variables, while the log-normal model can accommodate continuous response variable (Crow & Shimizu 1988). Log-Normal model has a continuous probability distribution of a random variable whose logarithm is normally distributed. The nature of crashes is random and variance is greater than the mean. In addition, crash data are not discrete but count data. That is why this model may fit better than the Negative Binomial (NB) model. The general form of the log normal model is given by Equation (4) (Crow & Shimizu 1987):

\[ \ln(Y) = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \ldots + \beta_n X_n \]  

where,

\( Y \) : observed crash count during a time period for site \( i \)

\( X_1, X_2 \ldots X_n \) : a series of variables, such as shoulder width, truck percentage etc.

\( \beta_0, \beta_1, \beta_2, \ldots \beta_n \) : regression coefficients to be estimated.

3.3 Model Selection

The model goodness of fit was examined using Akaike Information Criterion (AIC) and log likelihood values. The general equation of AIC is given by Equation (5) (Snipes & Taylor 2014).

\[ AIC = 2K - 2 \log(\text{likelihood}) \]  

where, \( K \) is the number of estimable parameters (degrees of freedom).

3.4 Cross-Sectional Studies

Different types of cross-sectional studies have been adopted to evaluate the safety effectiveness of countermeasures over the years. The HSM suggests to use regression models to compare the crash frequencies or rates between sites with and without a safety countermeasure. Cross-sectional studies do not need the time of the intervention of a treatment, which is considered one of the most important advantages for this method (Sasidharan 2011). Cross-sectional studies have two main steps (Tarko et al. 1998): 1) Develop a predictive model, and 2) Quantify the safety impacts of highway improvements.

To determine the safety effectiveness of a treatment, the Odds Ratio (OR) is calculated to assess the relative crash risk involving treatment sites and reference sites.

As mentioned earlier, many models and functional forms have been proposed to predict crashes in this study. For example, for NB models, as given in Equation (6).
\[ Y_i = \exp(\beta_0 + \beta_1 X_1 + \beta_2 X_2 + \cdots + \beta_n X_n) \]  

(6)

where,

- \( Y_i \) = observed crash count during a period for site \( i \);
- \( X_n \) = a series of variables, such as degree of curvature of site \( i \);
- \( \beta_0, \beta_1, \beta_2, \ldots, \beta_n \) = regression coefficients to be estimated.

Once the model is fitted and coefficients are estimated using observed crash data, the crash modification factor (CMF) for variable \( n \) can be then derived as shown in Equation (7).

\[ \text{CMF} = \exp(\beta_n) \]  

(7)

### 3.5 Before-after with Empirical Bayes

The observational before-after with Empirical Bayes (EB) method was introduced by Hauer (1997) (Hauer 1997). One of the main advantages of this method is that it accounts for changes in the crash frequencies in the ‘before’ and in ‘after’ periods at the treatment sites that may be because of regression to the mean bias (RTM). Therefore, this method is considered more accurate. However, exact implementation date of the countermeasure is one of the most challenging information to collect. This method is also considered a better approach than comparison group as it accounts for the influence of traffic volumes and time trends on safety (Schumaker et al. 2017).

There are three fundamental underlying assumptions in this method: 1) Crash frequency follows a Poisson distribution, 2) Gamma distribution of means for a population of systems, and 3) changes from year to year are similar for all reference sites. This method has 14 steps to calibrate Crash Modification Factors (CMFs) (AASHTO 2010). In this study, before-after with EB was also utilized to calibrate CMFs for shoulder rumble strips and compared with CMFs from cross-sectional analysis.

The estimate of the expected crashes at treatment sites is based on a weighted average of information from treatment and reference sites as given in Equation (8) (Hauer 1997):

\[ \hat{E}_i = (\gamma_i \times y_i \times n) + (1 - \gamma_i)\eta_i \]  

(8)

Where \( \gamma_i \) is a weight factor estimated from the over-dispersion parameter from the Negative Binomial regression relationship and the expected ‘before’ period crash frequency for the treatment sites as shown in Equation (9):

\[ \gamma_i = \frac{1}{1 + k + y_i \times n} \]  

(9)

\( y_i \) = number of the expected crashes of given type per year estimated from the SPF,

\( \eta_i \) = observed number of crashes at the treatment site during the ‘before’ period,

\( n \) = Number of years in the before period, and

\( k \) = over-dispersion parameter.

A typical SPF was mentioned in Equation (6). In this study, SPFs were developed as an output for the evidence of the effects of geometric and traffic characteristics from the reference sites. The overdispersion in the Negative Binomial model and the scale in the log-normal model indicate the level of widely dispersion of crashes around the mean. It should be noted that the estimates obtained from Equation (8) are the estimates for number of crashes in the before period. Since it is required to get the estimated number of crashes at the treatment site in the after period, the estimates obtained from Equation 16 are to be adjusted for traffic volume changes and different before and after periods. The adjustment factors for which are given as Equation (10).

Adjustment for AADT (\( \rho_{\text{AADT}} \)):

\[ \rho_{\text{AADT}} = \frac{\text{AADT}_{\text{after}}^{\alpha_1}}{\text{AADT}_{\text{before}}^{\alpha_1}} \]  

(10)

where, \( \text{AADT}_{\text{after}} \) = AADT in the after period at the treatment site, \( \text{AADT}_{\text{before}} \) = AADT in the before period at the treatment site, and \( \alpha_1 \) = Regression coefficient of AADT from the SPF.

Adjustment for different before-after periods (\( \rho_{\text{time}} \)) is given by Equation (11).

\[ \rho_{\text{time}} = \frac{m}{n} \]  

(11)

where, \( m \) = number of years in the after period, and \( n \) = number of years in the before period.

Final estimated number of crashes at the treatment location in the after period (\( \hat{E}_i \)) after adjusting for traffic volume changes and different time periods is given by Equation (12).
\[ \hat{\pi}_i = \hat{E}_i \times \rho_{AADT} \times \rho_{time} \]  

(12)

The index of effectiveness ($\theta_i$) of the treatment is given by Equation (13).

\[ \hat{\theta}_i = \frac{\hat{\lambda}_i / \pi_i}{1 + \sigma_i^2 / \pi_i^2} \]  

(13)

where, $\hat{\lambda}_i$ = observed number of crashes at the treatment site during the after period. The percentage reduction ($\bar{\theta}_i$) in crashes of particular type at each site $i$ is given by Equation (14).

\[ \bar{\theta}_i = (1 - \hat{\theta}_i) \times 100\% \]  

(14)

The odds ratio is given by Equation (15).

\[ \hat{\theta} = \frac{\sum_{i=1}^{m} \pi_i}{\sum_{i=1}^{m} \pi_i \times \var(\sum_{i=1}^{m} \pi_i)^2} \]  

(15)

where, $m$ = total number of treated sites and the variance of $\pi_i$ can be calculated from Equation (16) by Hauer (1997) (Hauer 1997).

\[ \var\left(\sum_{i=1}^{k} \pi_i\right) = \sum_{i=1}^{k} \rho_{AADT}^2 \times \rho_{time}^2 \times \var(\hat{E}_i) \]  

(16)

The standard deviation ($\hat{\sigma}$) of the overall effectiveness can be estimated using information on the variance of the estimated and observed crashes, which is given by Equation (17).

\[ \hat{\sigma} = \sqrt{\left[\left(\var(\sum_{i=1}^{k} \hat{\pi}_i) / \left(\sum_{i=1}^{k} \hat{\pi}_i\right)^2\right) + \left(\var(\sum_{i=1}^{k} \hat{\lambda}_i) / \left(\sum_{i=1}^{k} \hat{\lambda}_i\right)^2\right)\right]} \]  

(17)

\[ \var\left(\sum_{i=1}^{k} \hat{\lambda}_i\right) = \sum_{i=1}^{k} \lambda_1 \times \rho_{time}^2 \times \var(\hat{E}_i) \]  

(18)

Equation (8) is used to estimate the expected number of crashes in the after period at the treatment sites. This estimated expected number of crashes are compared with the observed number of crashes at the treatment sites in the after period to get the percentage reduction in number of crashes resulting from the treatment.

4. RESULTS AND DISCUSSION

4.1 Safety Performance Functions (SPFs)

Each predictive model is specific to a facility or site type for several years. It should be noted that the predictive method can be used to predict crashes for past years based on observed AADT or for future years based on forecasted AADT. While the Highway Safety Manual uses Negative Binomial models only to develop SPFs, various models were attempted in this study, as mentioned earlier, to find out the best fit for the crash data. Among the models, Log-Normal (LN), and Negative Binomial (NB) models were superior in predicting crashes. The best model was chosen based on AIC value. A lower AIC value indicated a better model fit. Negative Binomial (NB) model outperformed other models for the initial larger dataset. The log-normal model fitted the data better than other models for the final dataset (excluding segments with overlay and no SRS). The models are provided in Table 4.
For initial larger dataset, natural log of vehicle kilometers traveled (VKT) and degree of curvature were found to be significant to increase total crashes by 133 percent and 1 percent respectively at 95 percent confidence level. Truck percentage was also found to be significant at 95 percent confidence level for total crashes. Although degree of curvature was not a significant variable to predict fatal and injury crashes, other variables like vertical grades, average number of rainy and snowy days were found significant along with natural log of vehicle kilometers traveled and truck percentage. Lane departure crash is decreased by 4 percent with the increase of 1 meter in shoulder width at 90 percent confidence level.

**Table 4: Variable Estimates for Wyoming-Specific SPFs on Rural Two-Way Two-Lane Highways**

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total Crashes</td>
<td>F+I Crashes</td>
</tr>
<tr>
<td>Intercept</td>
<td>-3.9785</td>
<td>-5.8030</td>
</tr>
<tr>
<td>DOC</td>
<td>0.0089</td>
<td>-</td>
</tr>
<tr>
<td>VG1</td>
<td>-</td>
<td>-0.5305</td>
</tr>
<tr>
<td>VG2</td>
<td>-</td>
<td>-0.6176</td>
</tr>
<tr>
<td>VG3</td>
<td>-</td>
<td>-0.8325</td>
</tr>
<tr>
<td>SW</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Ln(VKT)</td>
<td>0.8458</td>
<td>0.9186</td>
</tr>
<tr>
<td>Truck</td>
<td>-0.0373</td>
<td>-0.0812</td>
</tr>
<tr>
<td>Rainy</td>
<td>-</td>
<td>-0.0132</td>
</tr>
<tr>
<td>Snowy</td>
<td>0.0016**</td>
<td>0.0045*</td>
</tr>
<tr>
<td>Dispersion/Scale</td>
<td>0.4058</td>
<td>0.5037</td>
</tr>
</tbody>
</table>

*Significant at 90% confidence level
**Significant at 85% confidence level

For the final dataset (excluding segments with overlay and no SRS), natural logarithm of vehicle kilometers traveled (VKT), vertical grades, shoulder width, truck percentage, and average number of rainy and snowy days per year were found to be statistically significant at 95 percent confidence level for both total and F+I crashes. Degree of curvature was found a significant variable along with significant variables for total crashes and fatal and injury crashes. This is because when the degree of curvature increases, lane maintenance becomes challenging. The results also indicated that steep downgrade increases total crashes. An increase of 1 meter shoulder width results in a 3 percent decrease in total and lane departure crashes and 8 percent decrease in F+I crashes. Park et al. (2015) also found a reduction in total and F+I crashes with the increase of shoulder width (Park & Abdel-Aty 2015). Vehicle kilometers traveled is mostly responsible for increasing number of crashes as it increases the exposure factor, which is in line with the literature (AASHTO 2010). Every 1 percent increase in the truck percentage, reduces 5, 5 and 6 percent total, lane departure and F+I crashes, respectively. A previous study showed that increasing percentage of trucks, lower the crash rate (Chalise 2016). This might be because of drivers being more cautious around large trucks. Average number of snowy days increases total, lane departure and F+I crashes while average number of rainy days reduces the crashes. Drivers usually get more cautious in both rainy and snowy conditions than normal weather condition. Hawkins (1988) found that reducing the vehicle speed, increasing the gap between vehicles and using warning signs decrease the crash frequency in rainy conditions (Hawkins 1988). However, in snowy conditions, drivers have less control over the vehicles on slippery roads resulting from black ice and blowing snows. Reducing the speed or using warning signs might not be useful to control crashes in this situation. (Hawkins 1988). An increase of one snowy day per year results in an increase of 2 percent of total crashes and 3 percent of F+I crashes. Saha et al. (2015) found crashes increase with the increase of snowy days in Wyoming (Saha et al. 2015).

**4.2 Cross-sectional Analysis**

Cross-sectional analysis was carried out utilizing shoulder rumble strips as a categorical variable with ‘no shoulder rumble strips’ as reference category. Similar to the Safety Performance Functions, Negative Binomial and Log Normal fitted better than other models for initial and final dataset respectively. The models for cross-sectional analysis are provided in Table 5.
### Table 5: Variable Estimates for Cross-sectional analysis

<table>
<thead>
<tr>
<th>Variable</th>
<th>Without Considering Intermittent Implementation of SRS</th>
<th>Considering the Intermittent Implementation of SRS (Continuous Implementation)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total Crashes F+I Crashes Lane Departure Crashes</td>
<td>Total Crashes F+I Crashes Lane Departure Crashes</td>
</tr>
<tr>
<td>Intercept</td>
<td>-5.3161 -5.5238 -4.4283</td>
<td>-6.5224 -9.7858 -7.6811</td>
</tr>
<tr>
<td>DOC</td>
<td>0.0083</td>
<td>0.0175 0.0116</td>
</tr>
<tr>
<td>VG1</td>
<td>-0.5324 -0.3421</td>
<td>0.3740 - 0.0946 -0.0389</td>
</tr>
<tr>
<td>VG2</td>
<td>-0.6122</td>
<td>-0.2300 -0.7869</td>
</tr>
<tr>
<td>VG3</td>
<td>-0.8571</td>
<td>- - -</td>
</tr>
<tr>
<td>SW</td>
<td>-0.0347** -0.0507</td>
<td>-0.0412 -0.0946 -0.0389</td>
</tr>
<tr>
<td>Ln(VKT)</td>
<td>0.7233 0.8485</td>
<td>0.9894 1.1809 1.1162</td>
</tr>
<tr>
<td>Truck</td>
<td>- - -0.0020 -0.0129</td>
<td>-0.0505 - 0.0096 -0.0111</td>
</tr>
<tr>
<td>Rainy</td>
<td>0.0056 0.0035</td>
<td>0.0145 - -0.0187</td>
</tr>
<tr>
<td>Snowy</td>
<td>0.0025* 0.0146</td>
<td>0.0649*** -0.2795 -0.2098</td>
</tr>
<tr>
<td>SRS</td>
<td>-0.0461** -0.1465</td>
<td>-0.3112 -0.2098</td>
</tr>
<tr>
<td>Disp/Scale</td>
<td>0.1977 0.4957</td>
<td>0.4396 0.2432 0.1390</td>
</tr>
</tbody>
</table>

*Significant at 90% confidence level
**Significant at 85% confidence level
*** Not significant

### 4.3 Crash Modification Factors (CMFs)

In this study, both cross-sectional and before-after with EB analysis were utilized to quantify the safety effectiveness of shoulder rumble strips. Table 6 represents the safety effectiveness of shoulder rumble strips for different crash types.

### Table 6: Safety Effectiveness of SRS in Rural Two-Way Two-Lane Highways

<table>
<thead>
<tr>
<th>Crash Type</th>
<th>Without Considering Intermittent Implementation of SRS</th>
<th>Considering the Intermittent Implementation of SRS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cross-section Before-After with EB Cross-section Before-After with EB Before-After for Individual Segments</td>
<td></td>
</tr>
<tr>
<td></td>
<td>CMF (Safety Effectiveness) S.E. CMF (Safety Effectiveness) S.E. CMF (Safety Effectiveness) S.E. CMF (Safety Effectiveness) S.E.</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>0.95 (5%) 0.11 1.40 (40%) 0.15 1.07 (-7%) 0.09 1.05 (-5%) 0.21 1.19 (-19%) 0.19</td>
<td></td>
</tr>
<tr>
<td>F+I</td>
<td>0.86* (14%) 0.19 0.84* (16%) 0.08 0.76* (24%) 0.14 0.45* (55%) 0.20 0.54* (46%) 0.19</td>
<td></td>
</tr>
<tr>
<td>Lane Departure</td>
<td>0.72* (28%) 0.10 0.79* (21%) 0.12 0.81* (19%) 0.10 0.58* (42%) 0.18 0.63* (37%) 0.13</td>
<td></td>
</tr>
</tbody>
</table>

* Significant at 95 percent confidence level

In general, there was a significant reduction in F+I and lane departure crashes but not in total crashes. These results are consistent with the literature (Torbic et al. 2009). Shoulder rumble strips are implemented to reduce lane departure crashes which result in severe crashes. Studies in Minnesota, Montana and Pennsylvania also concluded CMF for total crash as 1.18 (Torbic et al. 2009). In this study, before-after with EB analysis indicated more reduction in F+I and lane departure crashes compared to cross-sectional analysis. However, the reduction seemed to be more when the intermittent use of shoulder rumble strips was considered regardless the method of analysis. F+I crashes
were reduced by 14 and 16 percent for cross-sectional and before-after with EB analysis, respectively, when the intermittent use of shoulder rumble strips was not considered. Segments with continuous presence of SRS had 55 percent reduction in F+I crashes. Utilizing the EB analysis, F+I crashes are reduced 46 percent for before-after of individual sections which considers the intermittent nonexistent time periods of SRS as before period with after period when SRS are implemented. Lane departure crashes were reduced by 28 and 21 percent for cross-sectional and before-after with EB analysis without considering the intermittent use of shoulder rumble strips. Considering the intermittent use, the reduction for lane departure crash is 42 and 37 percent for continuous implementation and before-after for individual section, respectively, utilizing before-after with EB analysis. The results from this study are in line with previous studies for shoulder rumble strip. Torbic et. al (2009) reported from 10 percent reduction to 40 percent increase in all crashes after the installation of shoulder rumble strips in the NCHRP 641 report. The report also concluded an average 36 percent reduction in fatal and injury crashes for the installation of shoulder rumble strips. Studies in Missouri, Pennsylvania, Massachusetts, and British Columbia found 26, 33, 42 and 18 percent reduction in run off the road crashes, respectively. A previous study in Wyoming showed a reduction up to 29 percent in F+I crashes for shoulder rumble strips depending on different roadway facilities. The study used only a naïve before-after method with no consideration of the intermittent implementation of shoulder rumble strips. Results from this study is more reliable as the methods used in this analysis are more accurate and intermittent use was also taken care off in the analysis.

5. CONCLUSIONS

Shoulder rumble strips are considered an effective countermeasure to reduce fatal and injury crashes, especially in rural two-way two-lane highways. Most of fatal and injury crashes are the consequence of lane departure crashes. Lane departure crashes result in severe single vehicle run off the road crashes or opposite head on crashes. In some rural states such as Wyoming, when a new overlay is applied, the shoulder rumble strips are removed for extended period of time. Due to budgetary constraints and cost-effective project management strategies in Wyoming, there could be a long gap before re-implementing shoulder rumble strips. According to WYDOT, shoulder rumble strips will be removed for 2 years after implementing an overlay treatment. However, the time gap between removing SRS and reapplying them may vary. This study utilized an extensive data collection process to collect accurate dates when SRS were removed and reapplied. The researchers manually reduced SRS implementation utilizing Pathway Video Logs. Two possible approaches were adopted in this study to accurately assess the safety efficacy of SRS while considering the intermittent implementation issue: 1) the particular sections that had intermittent application, were excluded from the analysis (only segments with continuous SRS presence were considered), and 2) considering every off situation as before period and every on situation as an after period (before-after for individual segments). The results showed that the safety effectiveness of shoulder rumble strips was underestimated by both cross-sectional and before-after with EB methods before considering the intermittent implementation issue of shoulder rumble strips. Shoulder rumble strips resulted in about 50 and 40 percent in fatal and injury crashes and lane departure crashes, respectively, considering the intermittent use. When the intermittent use was not considered, the shoulder rumble strips yielded only 15 to 25 percent reduction in different type of crashes which might be misleading. By using an example from Wyoming, it can be concluded that transportation agencies should consider reapplying shoulder rumble strips right after an overlay to roadway segments to reduce fatal and injury crashes, and lane departure crashes. Moreover, the concept of the analysis presented in this study can be applied to other highways in Wyoming, as well as to other highways in other states experiencing the issue of intermittent implementation of shoulder rumble strips. The findings from this analysis can be used by transportation agencies across the world to evaluate and justify the safety improvements of rural two-way two-lane highways.

It is worth mentioning that the impact of resurfacing and overlay treatments is not explicitly considered in this study. The design of the study focused mostly on the impact of removing SRS on two-way two-lane highways.

ACKNOWLEDGEMENTS

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ABSTRACT:
The latest transportation technology is concerned with the commencement of driverless cars, whereas the third world countries are still striving for basic safety issues such as wearing a seatbelt. Although these countries have a very high rate of fatal and incapacitating crashes, the circumstances aren’t likely to improve much rather deteriorating. This paper aimed to observe the real-time scenario of seatbelt availability and use among vehicle drivers and occupants by collecting field data through in-person traffic survey in Dhaka city, Bangladesh. This study focused on seatbelt use in vehicles like buses, cars or trucks traveling long distances nationwide throughout the day, in which cases the crash and death rates are most severe. It also depicted the picture of seatbelt use in vehicles running in the capital city of Bangladesh where the most educated population dwells. This study found a very small percentage of drivers as well as riders use seatbelt while traveling short or long distances. surprisingly, it’s found that most of the public transport and freight trucks having high occupancy of people and goods traveling long distances in day and mostly night doesn’t possess seatbelts at all. A questionnaire survey among the travelers is also performed to sort out the causes behind not having/wearing a seatbelt. Lack of knowledge and self-consciousness, ignorance towards seatbelt use are found to be the most crucial factors of not having/wearing seatbelt. Finally, based on observation, potential solutions are suggested to introduce and increase seatbelt use among drivers and riders from the developing countries.
Real-Time Seatbelt Usage Scenario in Developing Countries Subjected to High Fatality of Crashes: An Observational Study of Dhaka City, Bangladesh

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1 INTRODUCTION

According to World Health Organization, road traffic injuries (RTI) are the number one cause of death among young people and the ninth leading cause of death across all age groups globally. Over 1.25 million people are documented to die each year followed by over 50 million people injured throughout the world due to traffic crashes. Over 90% of the total fatalities throughout the globe on the roads occur in low- and middle-income developing countries along with almost 5% loss of GDP because of these traffic fatalities although these countries possess only 54% of world’s vehicles. WHO defined five key risk factors for road traffic injuries – speed, drink–driving, and the failure to use helmets, seat-belts and child restraints. Among them, wearing a seatbelt alone can reduce the risk of fatality among drivers and front-seat passengers by 45–50%, and the risk of minor and serious injuries by 20–45% respectively. Among rear-seat passengers, seat belts reduce fatal and serious injuries by 25% and minor injuries by up to 75% (WHO 2016).

Bangladesh is a small South Asian country and one of the most densely populated countries in the world, has a traffic fatality rate of 64 per day which is the country’s number one cause of death among adults (Mohammad Al-Masum 2017). Unlike the developed countries, land transportation in Bangladesh is primarily based on publictransits i.e. public buses where the crashes are more frequent. This is one of the reasons why it has a very high crash and severity rate with very low vehicle ownership and vehicle mile travel (VMT). In spite of the high crash rate, there was no nationwide specific seatbelt law until 2015 (WHO 2015).

The main objective of this paper is to assess the seatbelt use rate amongst drivers and passengers of cars and buses traveling a long distance in Bangladesh as a representative of low-income developing countries. Field survey has been performed in the city of Dhaka, the capital of Bangladesh to find out the seatbelt usage along with a questionnaire survey to screen out the factors of not wearing a seatbelt. Also, several recommendations are advocated on how to increase the seatbelt use.

2 BACKGROUND AND LITERATURE REVIEW

The history of using seatbelt dates back in the early nineteenth century when some physicians in the US in the 1930s equipped their own cars with lap belts which pushed the manufacturers to include them in the vehicle design (Abbas et al. 2011). And the endeavor was fulfilled by Swedish inventor Nils Bohlin who developed the 3-point seatbelt to its modern form for Volvo—who introduced it in 1959 as standard equipment (Andréasson & Bäckström 2000). Numerous studies have been performed regarding the usage rate of the seatbelt by drivers and passengers and among different sex or age groups, behavioral or demographic factors of wearing seatbelts and way to improve seatbelt usage rate, effect of legislation change to usage rate etc. Lund A performed a study on voluntary seat belt use among U.S. drivers from 1982 data from 12 cities in the US (Lund 1986), Steptoe et al. examined trends in seat belt use by university students from 13 European countries between 1990 and 2000 (Steptoe et al. 2002), Huang et al. studied the attitudes and behavior of Chinese drivers regarding seatbelt use (Huang et al. 2011). Besides, studies had been done in different countries e.g. New Zealand (Blows et al. 2005) (Begg & Langley 2000), China (Passmore & Oзanne-Smith 2006), Mexico (Pérez-Nuñez et al. 2013), Nigeria (Sangowawa et al. 2010), Spain (Babio & Daponte-Codina 2006), Turkey (Özlem 2009), Malaysia (Ng et al. 2013) (Hauswald 1997), South Africa (Van Hoving et al. 2014), Egypt (Hoe et al. 2013), Qatar (Mahfoud et al. 2015) (Munk et al. 2008), Oman (McIlvenny et al. 2004), Saudi Arabia (Bendak 2005), UAE (Bendak & Al-Saleh 2013). But most of these studies are performed in developed and high-income
countries compared to little or no study in the low-income developing countries. For instance, there is no prior study about seatbelt use among vehicle users recorded in Bangladesh. Table 1 shows the rate of seatbelt use rate in some developed countries which indicated over 90% use of seatbelts in any user group.

Table 1. Seat belt use rates (%) in developed country

<table>
<thead>
<tr>
<th>Country</th>
<th>Seatbelt Use Rate (%) Among-</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Driver</td>
</tr>
<tr>
<td>Canada</td>
<td>92</td>
</tr>
<tr>
<td>Ireland</td>
<td>90</td>
</tr>
<tr>
<td>Japan</td>
<td>99</td>
</tr>
<tr>
<td>Sweden</td>
<td>98</td>
</tr>
<tr>
<td>United States</td>
<td>90.5</td>
</tr>
</tbody>
</table>


3 METHODOLOGY

Data have been collected through field survey at six locations of Dhaka city dated from February 5th to 15th 2017, on weekdays from 1:30 pm to 7:30 pm local time. Three of these locations are adjacent to main bus terminals of Dhaka city which are being used by the public buses traveling long distances from the capital city to distinct parts of the country throughout day and night. The rest three locations are also used by the long-range vehicles. They also serve as the entry or exit points of the city for public transits. Data were collected in person when the vehicles were stopped for passengers at terminals and at intersections when stopped at a signal. The target vehicles are randomly selected and inspected physically. Notes have been taken regarding the availability and usage of seatbelts among passengers and drivers specifically. A questionnaire survey is also performed among the occupants where feasible to find out the reasons for not wearing a seatbelt. Figure 1 shows one of the six locations when crowded, which indicates the presence of buses on the highway and traffic congestion on the outskirts of Dhaka city. Table 2 shows the locations of the survey, vehicles counts, vehicle types along with the capacity of vehicles considered under this survey. There were 1011 vehicles in total surveyed under this study. Among them, 221 were passenger cars, 174 passenger vans and 616 were public buses. Among the buses, 56 were luxury buses only which are found to have seatbelts for all occupants. Extra emphasis is given on the public buses traveling on the national highway because nearly 42 percent fatal accidents occurred on the national highway network (Hoque et al. 2006) and almost 36 percent of total crashes are occurred by bus and minibus in Bangladesh (MANIRUZZAMAN & MITRA 2005).

Figure 1. Jatrabari, Dhaka- one of the six locations under the survey, when congested

Source: The Daily Star (Pankag 2015)
4 RESULTS

Data collected and analyzed to find out the availability of seatbelt in a vehicle, capacity, and occupants of the vehicle and the number of occupants wearing a seatbelt. Special significance was put on whether the driver wearing a seatbelt. Passenger cars and vans are found to having seatbelts for all occupants but among buses, almost 90% had a seatbelt for driver only and nearly 9% which are luxury buses had seatbelts for all occupants as demonstrated in Table 3. Table 4 illustrates the vehicle capacity, vehicle occupancy, and occupants’ seatbelt usage rate by vehicle type. The study found about 29% and 23% of total occupants (including the driver) of cars and vans respectively wear seatbelts whereas the percentage is less than one for bus occupants. Figure 2 shows the percentage of occupants wearing seatbelt and seatbelts available in vehicle. In overall, only 2.3% of total vehicle occupants (including the driver) wear seatbelts. The rates for driver only are 100%, 88% and 36.5% for car, van, and bus respectively with an overall rate of about 59% as shown in Table 5. For passengers only, no passenger wears a seat belt in car and bus with only 7.5% wearing seat belts in vans and the overall rate of passengers wearing a seatbelt is just 0.21% which tends to zero as depicted in Table 5. Figure 3 delineates the percentage of driver and passenger waering seatbelt

Table 3: Availability of seatbelts in vehicle

<table>
<thead>
<tr>
<th>Vehicle type</th>
<th>No. of Seatbelt Available</th>
<th>N</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Car</td>
<td>All</td>
<td>221</td>
<td>100</td>
</tr>
<tr>
<td>Van</td>
<td>All</td>
<td>174</td>
<td>100</td>
</tr>
<tr>
<td>Bus</td>
<td>1</td>
<td>553</td>
<td>89.77</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>7</td>
<td>1.14</td>
</tr>
<tr>
<td></td>
<td>All</td>
<td>56</td>
<td>9.09</td>
</tr>
</tbody>
</table>

Table 4: Vehicle capacity, occupancy, and occupants’ seatbelt usage rate by vehicle type

<table>
<thead>
<tr>
<th>Vehicle type</th>
<th>N</th>
<th>Total Capacity</th>
<th>Average Capacity</th>
<th>Total Occupancy</th>
<th>Average Occupancy</th>
<th>Occupants Wearing Seatbelt (N)</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Car</td>
<td>221</td>
<td>1105</td>
<td>5</td>
<td>757</td>
<td>3.43</td>
<td>221</td>
<td>29.19</td>
</tr>
<tr>
<td>Van</td>
<td>174</td>
<td>1392</td>
<td>8</td>
<td>987</td>
<td>5.67</td>
<td>235</td>
<td>23.81</td>
</tr>
<tr>
<td>Bus</td>
<td>616</td>
<td>27075</td>
<td>44</td>
<td>27803</td>
<td>45</td>
<td>225</td>
<td>0.81</td>
</tr>
<tr>
<td>Total</td>
<td>1011</td>
<td>29572</td>
<td>29547</td>
<td></td>
<td></td>
<td>681</td>
<td>2.3</td>
</tr>
</tbody>
</table>

Total occupancy of buses is greater than total capacity because there were occupants standing in some buses.
Figure 2: Percentage of occupants wearing seatbelt and seatbelt available

Table 5: Driver and passenger seatbelt usage rate by vehicle type

<table>
<thead>
<tr>
<th>Vehicle type</th>
<th>Driver Wearing Seatbelt</th>
<th>Passenger Wearing Seatbelt</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>N</td>
<td>%</td>
</tr>
<tr>
<td>Car</td>
<td>221</td>
<td>100</td>
</tr>
<tr>
<td>Van</td>
<td>153</td>
<td>87.9</td>
</tr>
<tr>
<td>Bus</td>
<td>225</td>
<td>36.5</td>
</tr>
<tr>
<td>Total</td>
<td>599</td>
<td>59.25</td>
</tr>
</tbody>
</table>

Figure 3: Percentage of driver and passenger wearing seatbelt
5 FACTORS INFLUENCING LOW SEATBELT USE

From the result, it’s found that only 2.3% of total occupants of vehicles including the driver wear seatbelts which results in an additional risk of fatality in crashes. This is the scenario of the capital city of the country where the most educated population dwells, the situation is worse in the rural area. As part of the study, a questionnaire survey is also made to find out the reasons for not wearing a seatbelt. From the response of vehicle occupants, it’s found that almost 99% people are unaware or ignorant of the benefit of seat belt use and the fatal consequences of not wearing seatbelts. Lack of specific law and their enforcement is also found to be an important factor. Even until 2015, there was no specific nationwide seat-belt law and a law is proposed in 2017 which includes very small amount of fines for not wearing a seatbelt by driver only. The highway buses are more prone to severe crashes but only the luxury buses have seatbelts for all passengers which is only a negligible portion of total buses running throughout the country. From the result of this study, it’s found that only 9% of buses have seatbelts for all passengers. So, the absence of seatbelts in public buses is another vital factor for the low usage rate of seatbelts.

6 RECOMMENDATIONS TO INCREASE SEATBELT USE

The most effective way to increase the seatbelt usage among the drivers and passengers is the enactment of nationwide mandatory seatbelt usage laws (MULs) along with primary enforcement where some authorized personnel can cite a driver or passenger for not wearing a seat belt, without any other traffic offense being taken place. Study shows, the introduction of a seat belt law could result in a doubling or even tripling of belt use. For instance, seatbelt usage is increased by 28 percentage points after the law came into effect in developed countries like USA (Dee 1998). Although the first seatbelt use legislation was enacted in 1970 in Australia (Federal Office of Road Safety 1985) and even developing countries like India legislated the law in 1994, it’s very unfortunate for Bangladesh not having a seatbelt law. Although the enactment of law alone won’t be able to solve the problem unless the people are aware and conscious, the law would be able to create those consciousnesses. Another way to educate people and increase awareness towards the deadly effect of not wearing a seatbelt is seatbelt campaign and enforcement/publicity programs as they have potential to increase seatbelt use by 28% (Williams et al. 1987). The second important step to increase seatbelt use is to ban all the vehicles without seatbelt for all passengers from the national highway. Buses without seatbelt must not travel long distances or in highway rather they could be allowed within small city only. Finally, the responsibility and accountability of government authority and law enforcement agency could resolve this issue more than anyone else. At the same time, people should be self-aware for their own safety. Further research/study should be conducted on these issues to bring them into light. International organizations should come forward to help countries having the similar challenges.

7 CONCLUSIONS

Bangladesh, a low-income developing country is embedded with enormous challenges of demographic or socio-economic problems. Traffic safety is one the key issues which is responsible for an unexpected loss of lives everyday as well as GDP. So, the transportation flaws not only resulting in fatal and incapacitating crashes but also hindering the socio-economic growth of this nascent country. Because of very little research and under-reporting of situations, it failed to attract the attention of international community to come forward with solution and assistance. Also, political instability, corruption and state terrorism declined the standard of life and expectancy of people. So, diplomatic solution and political stability is required first before focusing on other issues.

Wearing a seatbelt is the foremost counter effect of the traffic fatality on roadways, but a very few portion of the total population in Bangladesh are aware of it and used to wear it. This study shows only 2.3% of total occupants wear a seatbelt and almost 90% public buses don’t have any seatbelt for their passengers. Furthermore, there is no established seatbelt law throughout the country even until now. An absence of specific law and enforcement along with public indifference over this issue made it more susceptible to fatalities. Law enactment and strict enforcement, raising public awareness, ensuring seatbelts to all vehicles running on the highways could be some alternatives to ameliorate this situation. The international organizations/communities such as UN, WHO could also contribute to solving this issue by providing counseling, guidelines and external pressures if applicable to the government to focus on the public safety rather than power and politics.
8 REFERENCES


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**ABSTRACT**

Roads code of ethics and conduct is a comprehensive program for enhancing drivers’ traffic safety culture and performance to establish a positive attitude toward safety on road by transforming the compliance of traffic safety from the result of external enforcement rules, regulations and punishment to an internal belief, value and culture. Roads code of ethics and conduct promotes six ethical values and six behavioral skills. Roads code of ethics and conduct is a proactive system because it treats the root causes of common unsafe behaviors for the strategic goal of “toward zero fatalities and serious injuries.”

Program consists of two main parts.

**Part I:** Six ethical values
- Responsibility
- Discipline
- Tolerance
- Respect
- Integrity
- Patience

**Part II:** Practical knowledge and skills
- Understanding traffic rules and regulations.
- Awareness of safety requirements by road users
- Awareness of vehicle safety tools and requirements.
- Awareness of the common causes of vehicle accident.
- Learning defensive driving skills
- Learning basic first aid and rescue skills

**Program Implementation:**
Comprehensive training book in Arabic and English versions is developed to meet the requirement of achieving the program objective through three main elements:
- foundation and definitions;
- scenarios and self-assessment; and
- adoption techniques.

**Measurement and Evaluation:**
Two online surveys is used (pre-and post-program) to measure two main areas of improvement:
- Knowledge and awareness; and
- Belief in

In addition to monitoring driver violations and motor vehicle accidents as key performance indicators (KPIs) for motor vehicle user compliance.
**Introduction and Objectives:**

Roads code of ethics and conduct is a comprehensive program for enhancing drivers' traffic safety culture and performance to establish a positive attitude toward safety on road by transforming the compliance of traffic safety from the result of external enforcement rules, regulations and punishment to an internal belief, value and culture (figure I). Roads code of ethics and conduct promotes six ethical values and six behavioral skills.

![Figure I](image)

Roads code of ethics and conduct is a proactive system that treats the root causes of the common unsafe behaviors with deep and sustainable impact for the strategic goal of “toward zero fatalities and serious injuries.”
Program contents

The program consists of two main parts.

- Part I: Ethical values
  - Foundations
  - Values based safety enhancement concept
  - Six ethical values (value meaning and why it is selected)
    I. Responsibility
    II. Discipline
    III. Tolerance
    IV. Respect
    V. Integrity
    VI. Patience

- Part II: Practical knowledge and skills
  I. Understanding traffic rules and regulations
  II. Awareness of safety requirements to road users
  III. Awareness of the vehicle safety tools and requirements
  IV. Awareness of the common causes of vehicle accidents
  V. Learning defensive driving skills
  VI. Learning basic first aid and rescue skills

Figure II
Brief of Part One:

- **Foundation**

The Values Based Safety Process. Terry McSween [https://g.co/kgs/bPRAcU](https://g.co/kgs/bPRAcU)

Most motor vehicle accidents are caused by unsafe acts by humans, so behavioral enhancement programs are very important in traffic safety strategies since drivers are not isolated individually. Their social lives should be included in the analysis of how they think and behave along with traffic rules, systems and regulations.

Behind each unsafe act are hidden motivators, and behind those motivators are deep causes and strong factors in each human personality such as driver habits, attitudes, paradigms, beliefs and values (Figure III).

![Figure III](image)

Treating unsafe behaviors from the deep level of personal values treats the root causes by highlighting and promoting ethical values and behavioral skills related to safe behaviors on the roads.

Six ethical values—discipline, responsibility, tolerance, respect, integrity and patience (DR-TRIP)—have been selected because they are the most dominant values that are most related to traffic safety behaviors.
Value definition and reason of selection

I. **Responsibility:**
What does it mean?
The state or fact of having a duty to deal with something or of having control over someone.
Why is it required?
Every driver should know that he has the ability of choosing his acts and reactions. Hence, he is responsible for all his behaviors and their consequences.

II. **Discipline:**
What does it mean?
The ability to do what one knows he should do, whether he feels like it or not.
Why is it required?
Driver discipline value is the main element to safety on the roads because failure to obey traffic rules and road regulation probably lead to accidents.

III. **Tolerance:**
What does it mean?
Tolerance is about accepting the fact that you cannot have your way in all circumstances in life or the ability or willingness to tolerate something, in particular the existence of opinions or behavior that one does not necessarily agree with.
Why is it required?
Yielding is one of tolerance value application, and failure to yield in traffic is one of the major causes of accidents in intersections before merging into another traffic lane or to a pedestrian or a bicyclist. Some drivers operate as if they always have the right of way.

IV. **Respect:**
What does it mean?
Respect means showing regard and appreciation for the worth of someone or something.
Respect involves considering others and treating everyone with dignity.
Why is it required?
Lack of respect toward others who are sharing the road or to traffic rules and regulation is the root cause of so many unsafe behaviors that lead to accidents’ such as tailgating, negligent driving, speeding, reckless driving, running red lights, running stop signs, unsafe lane changes and street racing.

V. **Integrity:**
What does it mean?
Integrity is aligning one's actions with one's stated values regardless of the situation.
Integrity is the commitment to do the right things and fight the temptation to compromise our values and beliefs.
Why is it required?
Drivers have to act safely everywhere regardless of whether the road is monitored or not. Otherwise, they will be hypocrites with two faces.

VI. **Patience:**
What does it mean?
Patience means to endure provocation, annoyance, trouble, delay, hardship, or pain with fortitude and calmness and without complaint, loss of temper, irritation or anger.
Patience means to hold inside temptation and motivators and outside challenges and activators from doing wrong things.
Why is it required?
Minor traffic infringements can sometimes lead to more serious incidents if drivers are not patient on the road.
Examples and scenarios (Figure IV):

It is well known that whenever we face a situation we react as shown in Figure IV, so between the event that we face and our reaction, there is a space of time in which there are so many factors that can control our reaction. The most effective factor is our personal values. If they are good and strong, all following reactions and behaviors will be good as well.

For example, the most common unsafe behaviors that always lead to vehicle accidents, especially in Gulf Areas, include like driving over the speed limit, using mobile phones while driving, not yielding to others, reckless driving, acting safely when the road is being monitored and acting differently when there is no monitor, and of road rage. All these unsafe behaviors will disappear if road users know and adopt DR-TRIP values.
Brief of Part Two:

Six practical knowledge and skills

I. Understanding traffic regulation
Traffic regulations are laws and rules that govern traffic, regulate vehicles and facilitate the orderly and timely flow of traffic. Knowing and following those rules and regulations will reduce the rate of accidents on the road and will the occurrence of traffic jams and delays in addition to protecting lives and properties.

II. Awareness of road users safety requirements
Everyone who travels on the road is a road user (Figure V), so understanding safety requirements is mandatory for all users, drivers, passengers, children, pedestrians and cyclists/motorcyclists.

III. Awareness of vehicle safety tools and requirements
Users need to know car safety tools and their ability to reduce the occurrence of vehicle accidents. Tools include seat belts, air bags, pre-crash systems, car seats, tires, car safety tringles, and first aid kits.

IV. Awareness of common causes of vehicle accidents
Understanding common causes of accidents will help drivers take preventative measures.

V. Learning defensive driving skills
Defensive driving is operating a vehicle in such a manner so as to create low risk to the driver and all other road users by constantly being aware of the surrounding situation. Defensive driving techniques focus on how drivers can overcome negative psychological factors such as stress, fatigue, emotional distress, distractions and road rage. It also offers instructions for developing a positive attitude behind the wheel by using proper judgment and increasing focus on the driving task. Defensive drivers can avoid crashes and help lower risk on the road.

VI. Learning basic first aid and rescue skills
Learning basic first aid and how to deal with victims during accidents is very important. It can mean the difference between life and death. Many accident victims can be helped if first aid is given before emergency services arrive.
Learning Approach

The learning approach for Part One and Part Two is to use the four stages of learning in shown in Figure VI.

*This learning stages model was developed by former GTI employee Noel Burch over 30 years ago.*

Program Implementation:

A comprehensive training book in Arabic and English versions is developed to achieve the objectives through three main elements:

- foundation and definitions;
- scenarios and self-assessment; and
- adoption techniques.

The facilitator who is conducting the training workshops should be carefully selected according to special criteria because he is key to the success of the program.
**Measurement and Evaluation:**

Two online surveys were developed as pre-and post program surveys to measure two main areas of improvement:

- knowledge and awareness; and
- believe in.

The program was conducted this year in April 2017 to around 80 university students. The survey measurement shows positive improvement in trainees knowledge and awareness from an average of 34% of correct answers in the pre-program survey to an average of 80% of correct answers in the post-program survey (Figure VII). An average of 95% of answers show strong belief in what they have learned with willingness to adopt and change behavior. The online survey is available via the links below.

Result of the Pre-Program Survey: [https://www.surveylegend.com/s/95z](https://www.surveylegend.com/s/95z)
Result of the Post-Program Survey: [https://www.surveylegend.com/s/90s](https://www.surveylegend.com/s/90s)

![Figure VII](image)

**Recommendations:**

- Governments should make understanding these code of ethics and skills one requirement for getting or renewing a driving license.

- It is recommended that Driving schools formally adopt By-law the Code of Ethics as a national standard.

- The program should be utilized by transport companies as mandatory training for drivers, including using individual surveys before and after the course and monitoring drivers' violations and motor vehicle accidents as KPIs for improvement toward zero fatalities and serious injuries and for saving the costs of vehicle damage, repair or violation penalties.
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- Program portal website in Arabic WWW.KSA-ROAD-Respect.com
Fatal Attraction: The Desire and Distractions Smart Phones Create on the Motorway

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KEYWORDS:
Distracted Driving; Smart Phones; Engineering Solutions; Temporary Portable Rumble Strip, Work Zones
Fatal Attraction: The Desire and Distractions Smart Phones Create on the Motor Way

Abstract

This paper addresses the growing menace of drivers using smart phones for texting and phone calls while driving. Using the latest research on the human brain, we will show the effects of texting on the driver’s situational awareness and reaction time. Reaction time is most critical as the driver approaches roadworks that require complex decisions to be made while the driver still has time to make the appropriate, safe maneuvers.

To reach the goal of “Zero Fatalities and Serious Injuries”, the paper will review the latest research on distracted driving looking at how use of smartphones detracts from driver performance. The paper will present an engineering based mitigation to distraction. To ensure that drivers are aware of approaching roadworks, especially those drivers using their smartphone for texting or phone conversation, many agencies deploy portable, temporary rumble strips. The paper will document best practice in the design and operation of roadworks utilizing these devices.

Introduction:

Each year, 1.25 million people die in traffic crashes worldwide. That’s equivalent to 1 death every 30 seconds. Another 50 million people are injured or disabled every year. Three elements contribute to this death toll: the roadway, the vehicle and the driver. Of these, design changes in both roadways and vehicles have made great strides in lowering the risk of crashes and death. The automotive industry has made many improvements in vehicle design like crumple zones, lane keeping technology and active cruise control. Road builders now install advanced safety features like the safety edge and crash attenuators making the roadway a lesser threat. That leaves us with the driver. An obvious danger is the impaired driver whether from alcohol or drugs. Also, excess speed and non-use of seat belts contribute to the high number of fatalities and injured. Safety professionals continue to improve strategies that are effective in reducing the numbers of impaired drivers on the roadways like the “Friends Don’t Let Friends Drive Drunk” campaign. Similar programs are raising the rate of seatbelt compliance to record levels.

Distracted driving, drivers using personal communication devices, has been a safety concern since the introduction of the first cell phone in 1983. The Motorola 8000X cost $3999 USD and provided 30 minutes talk time. Early cell phones had limited function and intermittent coverage so they were not widely considered a major safety threat. That threat increased significantly in 1993 when text capability was added to the phone. The next jump in function was the Sanyo camera-phone in 2002. With the introduction of the iPhone on June 29,2007, the handset in the car allowed the user to perform any task that could be done with an office computer. This improved functionality plus widespread coverage created an explosion in usage. In 1993 the subscriber base was estimated at 34 million users. By 2003 users had increased to 1.4 billion. By 2013, 4 billion people were using mobile phones. In 2016, that number grew to 4.6 billion meaning that 62.3% of the world’s population were using cell phones. [Statista, Inc. https://www.statista.com/statistics/274774/forecast-of-mobile-phone-users-worldwide/]

This explosion in smartphone use and the continued improvement in quality and strength of signal means that agencies will need to focus more resources on developing and implementing countermeasures. Before we look at a proven strategy, we need to define distraction and explore how the smartphone affects the driver’s ability to safely operate a vehicle.

Distraction generally takes 3 forms:

- Manual – any activity that removes the driver’s hand from the steering wheel. Activities include dialing a phone number, creating a text message, changing the station on the radio, adjusting the climate control system, eating a sandwich, etc.

- Visual – any activity in which the driver looks away from the road. Activities include reading a text message, reading an email, looking at the GPS display for directions, checking on children in the back seat, etc.
Cognitive – any activity that occupies the driver’s thought. Activities include composing a text or email message, a phone call, etc.

Operating a smartphone involves all 3 types of distraction. The option of hands-free can minimize the manual distraction. Likewise, voice recognition can minimize the visual distraction by permitting a driver to “dial” the phone with voice commands. However, the cognitive distraction remains prominent in any use of the smartphone.

Science has studied the effect of phone use on driving performance since at least the 1969 article by Brown, et al “Interference between concurrent tasks of driving and telephoning.” There have been at least 342 studies looking at 1608 measures of performance with 19370 subjects. The studies address a range of issues from the effect on the brain of conversation while driving, handheld versus handsfree phones and multi-tasking compared with task switching.

In “Cell Phone Induced Perceptual Impairments During Simulated Driving” Strayer et al. conclude that “active participation in a cell phone conversation disrupts performance by diverting attention to an engaging cognitive context other than the one immediately associated with driving.” Subjects missed twice as many simulated traffic signals and took longer to react to the signals that they did detect. Sonia Amado in “The effects of conversation attention and peripheral detection” [screen shot] compared the effect on a driver’s effectiveness of a conversation on the phone with a passenger in the vehicle and found little significant difference. However, when there is a dangerous condition, the passenger assists the driver to refocus on driving by ending the conversation while the caller is unaware and will continue speaking. Figure 1 uses Magnetic Resonance Imaging to indicate effect of conversation on brain activity in the motor control area. The image on the left has high activity when driving alone while the image on the right shows the decreased motor activity during conversation. The activity from the motor area has been transferred to the speech area in the frontal cortex.

Conversation impairs visual processing - fMRI

Figure 1. Conversation Impairs Visual Processing (Just et al. 2008)
Multitasking is often used to justify use of smartphones while driving. Dr. Steve Yantis published an article in “The Journal of Neuroscience” reinforcing earlier research on phones and driving. The brain does not so much perform two tasks simultaneously as switch back and forth, a process called task switching. “Directing attention to listening effectively ‘turns down the volume’ on input to the visual parts of the brain. The evidence we have right now strongly suggests that attention is strictly limited – a zero-sum game. When attention is deployed to one modality – say, in this case, talking on a cell phone – it necessarily extracts a cost on another modality – in this case, the visual task of driving.”

Conversation also affects driving performance by limiting the driver’s field of vision. In Figure 2 the red box outlines the area that a driver sees when not on a call. The driver can monitor oncoming traffic as well as potential pedestrians on the sidewalk. When in conversation this visual field shrinks both side-to-side and in the direction of travel. The driver would not be able to react in time should the van make an abrupt move into his lane.

![Conversation restricts visual processing - eye movements](image)

As the smartphones became more sophisticated, hands-free technologies like blue-tooth, seemed to be a way to make the use of cell phones safer by eliminating the manual distraction of holding the phone during conversations. Redelmeier et al. addressed handsfree calling in “Association between Cellular-Telephone Calls and Motor Vehicle Collisions” published in the New England Journal of Medicine. [reference] “We observed no safety advantage to hands-free as compared with hand-held telephones. This finding was not explained by imbalance in the subjects’ age education, socioeconomic status, or other demographic characteristics. … one possibility is that motor vehicle collisions result from a driver’s limitations with regard to attention rather than dexterity.”

Impaired driving from alcohol or drugs has been recognized as a prime contributor to the death toll on our roadways. One of the more disturbing outcomes of the focus on distracted driving is to see the parallels in distracted and drunk driving. Strayer et al. in the paper “A Comparison of the Cell Phone Driver and the Drunk Driver” worked to confirm earlier studies that driving while on a call was as risky as driving at the legal limit of alcohol. They did a
controlled experiment in a driving simulator with subjects at the limit of 0.08 wt/vol. performing the normal tasks of
driving. On the following day, the subjects drove in the simulator performing the same tasks in three sessions:
session 1 with no phone calls was used as the baseline. In session 2 the subjects used a handheld phone while in
session 3 they used a handsfree set. In their conclusion, they noted that the performance of the drunk drivers and the
cell-phone drivers differed from the baseline. The cell-phone drivers reacted slower to challenges in the simulation
and were more likely to crash. The intoxicated drivers were more aggressive in that they followed closer and did
braking with more force. “In the case of the cell phone driver, the impairments are due, in large part, to the
diversion of attention from the processing of information necessary for the safe operation of a motor vehicle. These
attention-related deficits are relatively transient [i.e., occurring while the driver is on the cell phone], whereas the
impairment from alcohol persists for prolonged periods of time.”

Dr. Strayer has provided a clue on how to diminish the risk of Distracted Driving: create a situation where the driver
puts down the cell phone. Now there is a method to mitigate distracted driving from one of the most dangerous
situations: approaching the road works.

Engineering Solution to Distracted Driving – Temporary Portable Rumble Strip [TPRS]

Temporary Portable Rumble Strip (TPRS) is a traffic safety countermeasure designed to reduce crashes and save
lives in construction work zones. TPRS is a transverse rumble strip, installed perpendicular to travel, and placed
well before the upcoming change in road condition, like a work zone. Drivers and passengers feel the significant
vibrations, and hear the loud, humming sound, when the vehicle passes over an array of strips. Distracted drivers re-
focus their attention on driving.

TPRS is not attached to the surface with fasteners or adhesive so that it may be deployed at the beginning of the
work period and removed at the end of the shift.

University of Kansas Study: Sound and Vibration

The Department of Transportation of the State of Kansas saw the emerging technology of TPRS as a potential
solution to their problems with distracted drivers ignoring the signage indicating an approaching work zone with the
result of too frequent multi-fatality crashes. To understand if TPRS could deliver an effective audible and vibratory
signal to the driver, the Kansas DOT contracted with the University of Kansas Engineering Department to evaluate
current and recently developed rumble strips. In 2009 KU presented the research paper, “CLOSED COURSE TEST
AND ANALYSIS OF VIBRATION AND SOUND GENERATED BY TEMPORARY RUMBLE STRIPS FOR
SHORT TERM WORK ZONES” describing their study of the TPRS at the Transportation Research Board in
January 2010.

KU Researchers tested the RoadQuake, an engineered polymer which is placed on the road surface without fasteners
or adhesive, and the Swarco Rumbler, a pavement marking polymer applied to the surface with adhesive. They used
permanent, in-pavement rumble strips that are grooved into the surface which is a practice used on rural roadways in
advance of stop signs as the benchmark for acceptable performance. Their measurements showed that RoadQuake
TPRS generated more vibration than adhesive rumble strips, and matched vibration levels generated by the
permanent, milled rumble strips. Similarly, they found that the sound levels of RoadQuake TPRS compared
favorably to permanent, milled rumble strips.
Specification Development

In a more recent report from the University of Kansas, Report KU-14-6, “Development of Temporary Rumble Strip Specifications”, commissioned by Kansas Department of Transportation, authors Steven D. Schrock, Ph.D., P.E., F.I.T.E, Vishal Sarikonda and Eric J. Fitzsimmons, Ph.D. add significantly to the body of knowledge of temporary portable rumble strips (TRS). Their report provides government agencies and manufacturers with viable specifications and standards, and more importantly, a system by which agencies can classify TRS devices by performance. This will ensure that the product will be both effective and stable under traffic conditions.

To develop these specifications and objective, the authors:

- conducted a review of previous studies about TRS
- reviewed practices of those state Departments of Transportation (DOT) that deploy TRS
- tested all commercially available devices, for speed and movement, on a closed course
- collected sound and vibration data for cut-in-place (CIP) rumble strips
- compared that data to sound and vibration data for temporary strips

In addition, the authors conducted testing of the strips stability under traffic. Using a passenger car and a heavy-duty truck, the traversed arrays of both strips submitted for testing at speeds from 22.5 MPH to 67.5 MPH. They monitored both the rotational and linear movement of the strips after each pass. Using that data, they established thresholds of performance based on speed. This data was used to compile a matrix with 4 classes of performance, again based on speed.

From this data, the authors created a Decision Matrix, which can be used by government agencies to approve and specify the use of commercially available TRS devices, based on test results and performance class of the TRS.

The key elements of a TRS include 1) that it needs no fasteners or adhesive, 2) that a 2-man crew be able to deploy and 3) that it be able to work in temperatures from -17°C to +80°C.
The authors established four classes of TRS, 1 through 4, with Class 1 the highest. Again, the classes correspond to interval speeds.

The authors explained the Decision Matrix Class 1 Classification:

“[A] Class 1 temporary rumble strip can be used at work zones whose speed limit is between 57.5 and 67.5 mph, irrespective of the [traffic] volume…. [and]…can be used on roads with volumes of AADT or ADTT exceeding 10,000 and 2,000, respectively, irrespective of the speed of the roadway.”

-Schrock et al, Page 46

Class 1 TRS, then, can be used in all work zones conditions shown in the Decision Matrix.

From their tests, the authors rated PSS RoadQuake 2F Temporary Portable Rumble Strip a Class 1 device, as linear and rotational movement were all within thresholds for Class 1. No other rumble strip tested met Class 1 specifications.
Best Practice in Use of TPRS:

The TPRS system has been in use since 2009. In that time practitioners have accumulated a significant body of practice that may be of value to agencies looking to improve safety of their work zones. This practice includes the design of arrays, the deployment techniques and the monitoring and movement of the strips.

Spacing Between Strips:

Stability under traffic is critical for optimal performance of the TPRS as the strips are not attached to the surface. The spacing between individual strips is critical to stability. To obtain optimal sound and vibration while maintaining stability, spacing width depends upon posted speed limits. Simply, the faster the speed, the wider the spacing needs to be, as shown in the graphic below.

![Figure 6. Recommended Spacing between Strips (PSS, 2015)](image)

Placement of TRPS Arrays in Work Zones:

After hundreds of experiments and tests of different array configurations and with support and confirmation from several university studies, industry has determined that 3 individual strips per array will alert drivers with sufficient sound and vibration so that they refocus their attention on driving.

Further, optimum performance results from the placement of 2 arrays per travel direction, in advance of a work zone. Two arrays will sufficiently alert drivers, especially distracted drivers, to the upcoming work zone. In the drawing above, which shows 1 direction only, 2 arrays are placed in advance of the work area.

The 2nd array warns drivers that
- The 1st array is not debris on the road
- The 1st array has been deployed intentionally
- The driver will soon approach the work zone
- The driver must soon take action
- The driver should not accelerate as they approach the work zone

The 2nd array also warns workers on-site, especially flaggers, that traffic is approaching.
Movement and Maintenance:

TPRS, as a temporary device that is not attached to the surface, will move from its original location when subjected to significant daily traffic, a mix of the types of vehicles, high work zone speeds, uneven road surfaces and slope. All these variables contribute to movement.

There are 3 types of movement to consider:
- **Skewing:** strip deviates from a straight line
- **Lateral:** strip moves side-to-side
- **Perpendicular to Travel:** strips move as an array in direction of traffic or opposite.

As an example, Figure 8 below shows the maximum allowance for skewing movement:

As TPRS is a temporary traffic control device, arrays should be monitored like all other devices. TPRS should be returned to their original position if they move more than allowed. PSS recommends that arrays are monitored at least every 4 hours.

TPRS will lose their effectiveness if the queue builds downstream of their location. Users should monitor for any queue build-up, and recalculate a new location for TPRS arrays.
Because TPRS may weigh up to 47 KG, at least 2 workers are required to deploy the strips manually. To diminish risk of soft tissue injury, many agencies are using devices to minimize the lift required to deploy and remove the strips. Below is a truck mounted device manufactured by Senn Konstruktionswerkstätte AG of Basel, Switzerland. To the right is the CRIB System manufactured in Cleveland, Ohio by PSS.

Figure 9. Senn Device on left, PSS RoadQuake CRIB Cargo Carrier and Retrieval System (PSS, 2016)

The Senn Device will install from the cab while the CRIB has the worker slide, not lift, the TPRS from the carrier.

Case Study: TPRS on a rural 2-lane 2-way

US 50 is a 2-lane, 2-way highway and a major east-west route through the center of the State of Kansas. In the spring of 2004, Shears & Co. was repaving a 25-mile segment of US 50 centered on the town of Peabody, Kansas. During the paving operation, all traffic was routed onto the open lane. Traffic was stopped at either end of the project while a pilot car guided the queue through the work area. On May 29, 2004, a tractor-trailer crashed into the queue waiting for the pilot car. The truck struck a passenger van ejecting both the driver and front seat passenger. Both were declared dead at the scene. On June 28th, another tractor-trailer struck a pickup stopped in the queue. The pickup burst into flames killing its 2 occupants. 2 days later a third tractor-trailer crashed into the queue killing 5 people in 2 vehicles including an 18-year-old pregnant woman. All 3 truck drivers were injured but all 3 survived. All were tested for alcohol and drugs and found negative. One driver took his eyes off the road to locate a pencil, another was fatigued due to lack of rest. All said that they did not see the full set of traffic control signs in advance of the lane closure.

Seven years later this section of US 50 was due to be repaved. The same contractor, now APAC Shears a LafargeHolcim company, won the contract. Working with the Kansas DOT, APAC developed a unique traffic control plan to address the risk of a repeat of the 2004 crashes. Kansas DOT’s maintenance forces had been using the RoadQuake portable rumble strip for work projects with a duration of a single shift. Their practice was to deploy the strips in 2 arrays of 3 strips each. The staging of the paving work presented a special situation: the length of the work area. APAC intended to pave up to 7 lane-miles per day. If they set the lane closure to match the day’s paving, they would exceed the DOT’s maximum length of 2 miles. APAC chose to preset 2 complete advance sign packages consisting of all required warning signs and the rumble strip arrays. The first package was set at the point where paving began that day. The second set was placed at the midpoint of the day’s paving in a ditch on the side of the road. This plan allowed APAC to continue the paving operation without having to stop to reposition the advanced signing. During the prepositioning of the rumble strips, care was given to place the strips upstream of where the queue was estimated to occur that day.

Because of this innovative use of temporary rumble strips, APAC Shears completed the 5-month contract not only with zero fatalities but with zero crashes.

Case Study: TPRS on a high-volume, high-speed multi-lane divided highway
Interstate 35 is a primary north-south route from Laredo, Texas to Duluth, Minnesota. There is a high percentage of heavy vehicles on this highway as it carries the bulk of the commerce between Mexico and the Midwest. With average daily traffic counts from 50000 to 100000 vehicles, the original roadway with 2 lanes in each direction was significantly over capacity. Texas DOT decided to upgrade 96 miles of I 35 centering on Waco, TX. This upgrade to 3 lanes in each direction necessitated removing and replacing all the structures and on/off ramps. A total of 17 were issued with work expecting to take 5 years. Most work was done at night so that daytime traffic could be maintained. No daytime lane closures were permitted. Queues created by the various merging tapers were particularly dangerous as drivers were not expecting closures in the rural section. In addition, the location of the merging tapers varied each night.

Working with researchers from Texas A&M University Transportation Institute, Texas DOT developed the End of Queue System. The system is composed of 2 parts: 1) speed detection devices that communicate in real time with portable, changeable message boards to let drivers know of downstream traffic conditions and 2) portable rumble strips that provide audible and vibratory warning of upcoming merging tapers. Two deployment schemes were developed to accommodate to queue lengths of 3.4 miles and longer queues of 7.2 miles. The End of Queue systems were removed each morning at the end of the work period.

In 2015, TTI researchers compared 234 nights of lane closures where the End of Queue was not deployed, with 216 nights where the EOQ was deployed to determine effectiveness of the system.

There was a 29% reduction in severe crashes, crashes resulting in serious injury, and a 38% reduction in rear-end crashes. Researchers then evaluated the societal savings of the reduction in crashes determining that each deployment resulted in a savings of $6316 per night of use.

In January 2017, TTI researchers, using Bluetooth technology to detect presence of queues, compared the reduction of crashes when the portable rumble strips alone were used with those times when the End of Queue system combined with portable rumble strips were deployed. Both options had a statistically significant effect on crash reduction. When used alone, the portable rumble strip was slightly more effective than when used in conjunction with the EOQ system. As part of this analysis, TTI looked at the reduction in crashes involving fatalities or severe injuries when the safety treatments were used in queueing situations. The 68% reduction in severe crashes confirmed a savings of $6600 to $15000 per night of deployment.
Conclusions:

Distracted driving, drivers using smartphones for calls and texts, are a growing threat to all road users. The number of smartphones in use will rise to 6.6 billion by 2020. Not only is the number of phones growing exponentially, the devices are more sophisticated meaning drivers spend even more time on their phones. Science shows that a driver's ability to respond to changes in the roadway, like an approaching work zone, drops significantly when the brain switches from visual to auditory processing. A driver using a smart phone is a driver more impaired than the same driver with 0.08% blood alcohol. The impairment of smartphone use ends as soon as the phone is put down. The recently introduced technology of TPRS sends a strong auditory and vibratory signal to the driver and has been shown to cause the driver to stop phone use.

The safety community is encouraged to develop improved techniques and programs to both increase drivers understanding of the dangers of smartphone use while driving and at the same time implement existing mitigations for those drivers that continue to drive and text.

References:


Assessment of Blackspots in a Hill road: A case study of Surkhet - Jumla road in Nepal.

Er Tulasi P Sitaula- Former Transport Secretary and practising Highway Engineer

Er Anga Lal Rokaya – Highway Engineer

Nepal in an attempt to enhance connectivity to its remote districts connected 74 of its 75 district headquarters by road access. The remaining one will be connected next year. Despite this achievement, many hill roads have severe problems with road geometry, road surfacing and frequent accidents. Surkhet- Jumla road (manma-Jumla section 82.6 km length) in North western part of Nepal, a hill road with harsh terrain, sharp bends, steep gradient and narrow width was studied for its geometry, accidents and black spots. This road is named by media as’ killer road’ due to huge numbers of fatality on this road over a very small period of time since its opening.

The Study assessed the safety situation, identified the reasons for accidents and recommended possible actions to enhance safety in hill roads. Ten black spots were found in this section and the road condition including poor geometry are responsible for 44% of the accidents that occurred here. If these black spots are not improved quickly the trend of increasing accidents will continue. Recommendation have been made to the concerned government department for improvement of this section on a phased and prioritised manner. This study is believed to help identify black spots in other hill roads as well. The results and recommendations could be applicable to other hill roads thus contributing towards reducing accidents and fatality and saving precious lives thus, supporting to meet the road safety target set by Nepal in line with the UN Decade of Action 2011-2020.

Key words: Black spots, hill road, enhancing safety

Background

Nepal is a mountainous and landlocked country. Composed of a land form of Plains, hills and mountains in the percentage of 17%, 66% and 17% respectively it has also a severe altitude variation from 100 m from mean sea level to 8848 m, the highest peak in the world. Interestingly, this happens in a narrow space of 180 kilometres bringing huge bio diversity. Nepal borders China to the north and India in remaining three directions. With a per capita income of less than 3 dollars a day, Nepal is an under developed country trying to graduate to developing country in the near future. Its enormous hydro power potentials are yet to be developed. Infrastructure is quite inadequate and very poor thereby hindering the economic development of the country. With a land size of 147181 sq. km and population of approx. 30 million Nepal lies in the middle in the list of countries in the world.

Nepal has developed some road infrastructures in the past with its focus on connecting district headquarters with motor access. This goal is almost fulfilled with only one out of 75 districts deprived till date. In the process of fast developing road access and everybody putting high priority on road quantity, the roads thus developed are unsafe for the traffic. The roads are having narrow width, steep gradients, poor surface and sharp un protected bends. The road safety elements are grossly forgotten on many of these hill roads. This has increased the rate of Road traffic accidents on hill roads of Nepal.

A study was conducted to identify the blackspots and to assess the road safety scenario in a critical section of a hill road named Surkhet -Jumla Road (karnali Highway). The objective of the study was:

- To identify the causes of road accident in Karnali Highway.
- To analyze the opinion of road users in Hilly roads about Road accidents.
- To analyze hazardous locations/black spots in Karnali Highway and suggest necessary improvements to enhance Road safety.

This paper highlights the results of this study.
The Project area

Karnali highway has very bad reputation in terms of road traffic accidents and is often called killer road by media and other stakeholders. The major complaint has been made on geometry of the highway because most of the accidents occurring here are connected to the poor geometry of the highway. So the study area is Manma-Jumla section of the highway, which passes through rocky and steep terrain, starting from Manma (Ch 154+000) having altitude of 1760 m and ends at Jumla bazaar (236+640) having altitude of 2370 m from Mean Sea level, MSL.

Map Showing Project area and Road alignment

Methodology of the study

The study comprised of field works as well as secondary data collection. The road geometry was measured in the field and opinion of road users and experts was collected during the field works by means of a set of structured questionnaires.

Field work phase

- Collection of primary and secondary data
- Distribution and collection of questionnaire survey
- Identification of major accident locations
- Finding of possible causes and identification of possible countermeasures

Secondary data collection

- Road Accident Data from secondary source: District police & local people
Traffic volume counts (AADT) of Manma -Jumla Road

Primary data collection

A) Data was collected from two types of respondents: Experts (Engineer) and Road users (Driver, passengers and other people)

B) From Road geometry survey- Visible dimensions of road & provided safety measures were noted down

Collection of data were grouped and analyzed in following areas:

• Rate of accidents in Manma – Jumla road section?
• Causes of unsafe travel in the road section?
• Road width, curve radius and extra widening situation in the study section?
• Gradients and sight distance provided in the study section?
• Feeling safe while travelling on the road section?
• Most accidents spots and time of accident?
• Main factors responsible for accident in the study area?

The Accident trend

Accident trend in Surkhet Jumla Road( Manma- Jumla section)

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<td>27</td>
<td>2.45</td>
<td>0.34</td>
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<tr>
<td>2014</td>
<td>17</td>
<td>23</td>
<td>11</td>
<td>2</td>
<td>57</td>
<td>32</td>
<td>1.39</td>
<td>0.68</td>
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<tr>
<td>2015</td>
<td>14</td>
<td>5</td>
<td>6</td>
<td>12</td>
<td>24</td>
<td>18</td>
<td>3.60</td>
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<td></td>
</tr>
<tr>
<td>2016*</td>
<td>4</td>
<td>1</td>
<td>1</td>
<td>9</td>
<td>17</td>
<td>7</td>
<td>7</td>
<td>0.13</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>61</td>
<td>88</td>
<td>57</td>
<td>83</td>
<td>256</td>
<td>140</td>
<td>6</td>
<td>0.58</td>
<td></td>
</tr>
</tbody>
</table>

Source for highlighted data : Traffic Police Kalikot and Jumla

2016* data are for first 3 months only.

F= fatal  S= Serious Injury  M= Minor Injury  P = Property damage

SI= severity Index

Analysis of Black spots

Black spots were verified using Critical Crash Rate Method.

Crash rate per 100 million vehicles kilometer (RMV) is given by the equation,
RMV = A \times 100000000/VT \quad \text{(Eq 1)}

Where,

A = \text{No of crashes, total at the study location during a given period}

VT = \text{vehicle kilometre of travel during the given period.}

VT = \text{ADT} \times \text{N} \times \text{S} \quad \text{(eq 2)}

Where,

\text{ADT} = \text{average daily traffic}

N = \text{no of days in study period}

And S = \text{length of road segment},

**B) Calculation of critical crash rate**

- The critical crash rate (CR) with 95 percent confidence was calculated by the equation:

\[ \text{CR} = \text{AVR} + 0.5/\text{TB} + \text{TF} \sqrt{\text{AVR}/\text{TB}} \quad \text{(Eq 3)} \]

Where,

\text{AVR} = \text{Average accident rate for category of highway being studied}

\text{TB} = \text{vehicles exposure at location}

\text{TF} = \text{test factor, standard deviation at a given confidence level} (\text{TF} = 1.96 \text{ for 95% confidence level})

**Comparison of crash rate and critical crash rate and ranking of locations**

The RMV values equal or greater than CR value = Black Spots.

The ranking or crash location was divided in four groups as

- Black Spots having \(\frac{\text{RMV}}{\text{CR}}\) value ≥1,
- risk Spots with 0.6-1,
- Moderate Risk Spots with 0.3-0.6 and
- low risk spots with \(\frac{\text{RMV}}{\text{CR}}\) value 0.0-0.3.

**Geometric elements Survey spots are as follows**

1. Manma east ch. 154+440
2. Daha, Kalikot Ch.161+500
3. Bhaisigauda, Chhapre VDC, Kalikot Ch. 185+700
4. Birtamode, Pakha VDC, Kalikot ch. 187+00
5. Timure Bheer, Jubitha VDC, Kalikot Ch. 188+500
6. Khalla Gad, Kalikot ch.192+500
7. Rachuli, Kalikot ch.197+00
Out of the eleven spots with high rate of traffic accidents analysed, ten spots confirmed to black spot criteria. The eleventh one was close to black spot threshold and is now classified as Risk spot. Among the ten black spots 8 spots have value of RMV/CR greater or equal to 2 showing the high risks associated with them. Birta mode, the first ranked black spot from this analysis is also felt most critical spot in the road by the road users as well.

**Causes of Accident**

The study showed that the primary cause of accident is deficient geometry. Among geometricaelf deficiency 55% is attributed to narrow width, 26% to small radius of curves, 16% to insufficient Sight distance and 3% to high gradient. 70% of the responding road users and engineers view that the road width provided is quite insufficient. Like wise 60% of respondent feel that extrawidening provided at curves is not sufficient. Sight distance is felt insufficient by 60% of respondents whereas 62% are not comfortable with the gradient provided in the road section.

The road users view that the road accident is caused due to poor road (44%), negligence of driver (41%), vehicle condition (8%), Negligence by pedestrians (5%) and road encroachment (2%). In the meantime engineers responded that the basic causes are Roads (58%), Drivers (25%)and encroachment (18%).

the involvement of vehicles by type in accident is that Passenger buses are 40%, Trucks 10%, Tractors 15%, Car/Jeep van 21% and Motor cycles 15%.

The drivers behaviour in the accident was found as drunk 50%, Young and unskilled 17%, old and skilled 17%, old and unskilled 8% and young and skilled 8%.

**Conclusion**

The following conclusions are drawn from the study:
The study section was found more hazardous for traffic safety requiring immediate improvement in the field of Road geometry.

The Whole road length needs to be paved with necessary safety measures.

Old vehicles in poor condition (60%) are found to ply on this road among which Buses are involved in (39%) of accident requiring quick attention of enforcing authority.

Accidents are found to occur mostly in night time and during rainy season.

Width of road, radius of curve and extra widening at curves in the study area is insufficient.

Two-third of the respondents are not happy with the road width whereas 82% of people travelling in Surkhet- Jumla road feel unsafe to travel here.

Large portion of respondents (43%) feel that accident is due to road condition followed by drivers’ fault (41%)

Respondents belief as well as black spot analysis and field survey shows that Birtamode in Pakha VDC having CCR value of 6.344 is the most severe location for accidents.

**Recommendations**

- Since the travelling public feel that Birtamode in Pakha VDC is the most accident prone zone in Surkhet- Jumla road, the Road authority should improve this location without further delay.

- Star rating of the entire length of Surkhet - Jumla road should be done using standard global rating practice.

- Black spots should be improved in the priority based on their rankings found in this study and on the basis of future star rating.

- The major causes of accidents as identified here should be tackled in a systematic and participatory manner by duly consulting stakeholders.

- As the road is constructed with poor geometry, the vehicles need to be new and of better condition with mature and experienced driver. Department of Transport Management and Traffic Police should be more careful to ensure this.

- As the accident happened mostly at night time and rainy season the night driving shall either be prohibited or be made only with awareness programs through education and enforcement

- Many of the black spot area have poor and unpaved road surface compounded by sharp bends and steep gradient. Appropriate surfacing should be done in the vicinity of these spots.

- Road furniture’s including edge line paintings, crash barriers and signals showing sharp turns and steep gradient are required to be put into place in these location as early as possible.

**References**


• Ambros, J. et al., 2016. Identification of hazardous locations in regional road network- comparision of reactive and proactive approaches.


• Chen, F. & Chen, S., 2013. *Differences in injury severity of accidents on mountainous highways and non mountainous highways*.

• Department of Road, Road statistics, 2014
Photos

Rachuli, Kalikot ch. 197+00 (2073-5-16)
Table showing geometric characteristics of the spots analysed

<table>
<thead>
<tr>
<th>Name</th>
<th>Chainage</th>
<th>Length of spot, m</th>
<th>Formation Width, m</th>
<th>Radius of curve, m</th>
<th>Safety measures</th>
<th>Gradient %</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manma east</td>
<td>154+590</td>
<td>320</td>
<td>4.3</td>
<td>15</td>
<td>None</td>
<td></td>
<td></td>
</tr>
<tr>
<td>daha</td>
<td>161+500</td>
<td>460</td>
<td>4.0</td>
<td>12</td>
<td>Some delineator posts only</td>
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<td></td>
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<tr>
<td>Bhaisi gauda</td>
<td>185+700</td>
<td>420</td>
<td>4.0</td>
<td>15</td>
<td>15</td>
<td>10</td>
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<tr>
<td>Birta mode</td>
<td>187+00</td>
<td>330</td>
<td>4.1</td>
<td>10</td>
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<td>10</td>
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<tr>
<td>Timure bheer</td>
<td>188+500</td>
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<td>4.3</td>
<td>15</td>
<td>none</td>
<td>6.5</td>
<td>Steep grade</td>
</tr>
<tr>
<td>Khalla gad</td>
<td>192+500</td>
<td>346</td>
<td>4.5</td>
<td>15</td>
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<td>8.5</td>
<td>Unpaved surface, bridge under construction</td>
</tr>
<tr>
<td>Rachuli</td>
<td>197+000</td>
<td>350</td>
<td>3.8</td>
<td>16</td>
<td>none</td>
<td>15</td>
<td>Steep, unpaved</td>
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<tr>
<td>Kudari</td>
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</table>
Investigating Safety Performance Function Transferability and Calibration

Towards zero fatalities and serious injuries

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KEYWORDS:
Include up to 5 keywords
Safety performance functions, transferability, negative binomial models, calibration, Highway Safety Manual

ABSTRACT:
Safety performance functions (SPF) are essential mathematical formulations for roadway agencies and traffic engineering consulting firms for estimating crash counts by roadway facility type and crash category. That is in order to be able to identify high frequency crash locations, otherwise known as network screening. Crashes may be classified by severity, type or both. The Highway Safety Manual (HSM), published by the American Association of State Highway and Transportation Officials (AASHTO), provides a series of SPFs for a variety of roadway facility types. Agencies may collect and process local crash and geometric data to develop own localized SPFs. Such agencies may also opt to borrow SPFs from neighboring jurisdictions or from the HSM. This cuts the costs of data collection and hiring experienced data analysts for SPF development. This research investigates the transferability of SPFs of total crashes at rural divided multilane highway segments among three states. They are Florida, California and Washington. In addition, the HSM provides a simple SPF calibration technique for agencies implementing and calibrating the HSM’s SPFs for their own jurisdictions. The technique involves assigning a single calibration factor to every site of the specific roadway facility of which the SPF is applied to predict crash counts. Hence, the HSM calibration technique is not accurate. In this research, a more recent SPF calibration technique in the traffic safety literature is applied for the three aforementioned states’ SPFs to enhance their transferabilities.
Investigating Safety Performance Function Transferability and Calibration

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INTRODUCTION

There were 17,114 fatalities resulting from traffic crashes at rural roads in the US in 2015. The number of fatal crashes at rural roadway facilities accounts for 49% of the total fatal crash counts in the nation even though only 19% of the population live in rural areas (Traffic Safety Facts 2015 Data Rural/Urban Comparison 2017). It is thus the traffic engineer’s responsibility to design safer roads by providing safety countermeasures such as rumble strips, wide shoulders, clear zones, delineators, crash attenuators and profile pavement markers among others. The Highway Safety Manual (HSM) provided by the American Association of State Highway and Transportation Officials (2010) specifies a procedure for identifying high frequency crash locations, also known as hot spots, and testing the efficacy of the appropriate safety countermeasures applied to the hot spots. Specifically, reduction in crashes at the hot spots may or may not necessarily be due to implementation of the countermeasures. Crash frequencies may be increasing or decreasing over time. Also, a possible trend observed in traffic crash data, known as regression to the mean, occurs when crash counts fluctuate around their mean. Thus, the HSM outlines and recommends the empirical Bayes (EB) method to check the effectiveness of the countermeasures in reducing crashes at hazardous sites while taking into consideration the crash trends. The EB method requires statistical regression models, called safety performance functions (SPFs), to estimate crash counts by roadway facility type, crash type, severity or a specific level of severity belonging to a crash type such as a rear-end property damage only crash. This requires collection of data of mainly crashes and road geometric characteristics even though weather conditions at the time of the crash, topographic characteristics and socio-demographic characteristics can also be collected. Once the data are collected, the crash counts per site are modeled as a function of the characteristics previously mentioned. Jurisdictional agencies may opt to develop own localized SPFs. Yet, agencies willing to cut costs of hiring expert data analysts to collect and process data, may choose to adopt SPFs from elsewhere or the HSM. Therefore, this research is aimed at investigating the transferability of SPFs of total crashes of three states, namely Florida, California and Washington. The SPFs are for rural divided multilane highway segments. Rural divided multilane highway segments are defined as bi-directional four-lane segments with median separators or two-way left-turn lanes with partial access control. The segments ought to be located outside areas classified as urban areas. The medians may or may not include barriers. Also, the HSM provides a technique to calibrate its SPFs to conditions of specific locations. In this research, the HSM calibration technique and a modified version of it (Srinivasan et al. 2016) are applied to calibrate the SPFs of the three states. In the following sections, discussions of the background of the topic of this research, a literature review, data preparation, research method, analyses and conclusions are provided.

BACKGROUND

Modeling crash counts using ordinary linear least squares regression is not appropriate since crashes are discrete and non-negative (Garber and Wu 2001). Count modeling approaches such as the Poisson and negative binomial (NB) models are apt for regressing crash counts. Yet, a typical factor observed in crash data is that the variance of the crash counts is greater than the mean, a condition known as overdispersion. Overdispersion is a direct violation of the Poisson model. The NB model is a variation of the Poisson model and is used as the conventional model for crash count estimation (Lord et al. 2005). In this research, NB models are implemented. Under the NB model, the probability of a crash at site, i, is \( p_i \), and is given as follows (Lord et al. 2008).

\[
p_i = \frac{\Gamma(y_i - \phi)}{\Gamma(\phi) \times y_i!} \times \left( \frac{\phi}{\mu_i + \phi} \right)^{\phi} \times \left( \frac{\mu_i}{\mu_i + \phi} \right)^{y_i}
\]

In Equation (1), \( y_i \) is the observed crash count at site \( i \), \( \phi \) is the inverse of the overdispersion parameter, \( k \), \( \mu_i \) is the mean crash count at the site \( i \) and \( \Gamma(.) \) is the gamma function. Note that the Poisson model is the limiting form of the NB model as \( \phi \rightarrow \infty \). The mean function, \( \mu_i \), is specified as shown in Equation (2).

\[
\mu_i = \exp\left[ \beta_0 + \beta_1 X_{i1} + \beta_2 X_{i2} + \beta_3 X_{i3} + \ldots + \beta_j X_{ij} \right]
\]

The mean, \( \mu_i \), is the crash count that is predicted by the SPF, known as \( N_{SPF} \). From what can be understood by Equation (2), the predicted crash count is a function of the roadway geometric design characteristics, traffic characteristics and...
other characteristics such as socio-demographics, \( X_i \)’s. The parameters, \( \beta \)’s, are regression coefficients obtained using maximum likelihood estimation (MLE), a statistical optimization technique.

As previously mentioned, the HSM provides a variety of SPFs by facility type and crash category. The HSM defines what is referred to as base conditions for every facility type in order to apply the HSM’s SPFs, provided. Since this research is focused on rural divided multilane highway segments, the HSM’s base conditions for rural divided segments are listed as follows.

- 12 ft (3.658 m) lanes
- 8 ft (2.438 m) paved right shoulders
- 30 ft (9.144 m) medians
- No automated speed enforcement
- No street lighting

Deviations from the base conditions are accounted for by crash modification factors (CMFs), also provided by the HSM. The general form of the HSM’s SPFs for rural divided multilane highway segments is the following.

\[
\hat{N}_{SPF,i} = \exp\left[\hat{A} + \hat{B} \times \ln(AADT_i) + \ln(L_i) \times \prod_{j=1}^{f} CMF_{i,j}\right]
\]  

(3)

The regression coefficients are renamed while the term, \( AADT \), is the annual average daily traffic (veh/day). The term, \( L \), is the roadway segment length (km) since the crash data are prepared by grouping crashes by homogenous segments. In each homogenous segment, the geometric characteristics are uniform. The CMFs are multiplicative factors, each of which is responsible for correcting for a specific deviation from the defined base conditions. The overdispersion function, which is a fundamental component of the NB model, is estimated as shown in Equation (4).

\[
k_{i} = \frac{1}{\exp[C + \ln(L_i)]}
\]  

(4)

In Equation (4), another parameter, \( C \), is estimated via MLE. For an agency, willing to apply the HSM to its own jurisdiction, the HSM provides a calibration factor technique. It is aimed at multiplying the right-hand side of Equation (3) by a calibration factor, \( C_{rd} \), defined as follows.

\[
C_{rd} = \frac{\sum_{i=1}^{N} N_{obs,i}}{\sum_{i=1}^{N} N_{SPF,i}}
\]  

(5)

The term, \( N_{obs,i} \), is the observed crash count at site \( i \). A calibration factor of 1 indicates that the HSM’s SPF perfectly predicts crashes at the jurisdiction to which it is being applied. The HSM calibration technique is subject to criticism because the factor, \( C_{rd} \), is generic and not site specific. That is, it can aggravate the prediction errors for some sites while shrinking the errors for others. A discussion of the literature is provided in the next section.

LITERATURE REVIEW

Few research efforts were made to investigate the transferability of SPFs across localities. In the majority of past studies, the HSM’s SPFs were calibrated to specific states and abroad the US. In addition, Srinivasan et al. (2016) developed an enhanced version of the HSM calibration method. The SPF calibration studies and Srinivasan et al.’s (2016) method are discussed. In addition, limitations of the past studies and this research’s contribution to the literature are discussed.

Sun et al. (2014) calibrated the HSM’s SPFs of total crashes to Missouri’s rural divided multilane highway segments while taking into account the CMFs. The calibration factor, obtained, was 0.98 indicating that the HSM’s SPFs marginally over-predicted crashes in Missouri. The research team repeated the analysis procedure for three-leg unsignalized intersections and four-leg unsignalized intersections. The calibration factors, computed, were less than 0.4. A possible explanation is that the crash records, sampled, were fewer than 100 which is not recommended by the HSM. Srinivasan and Carter (2011), undertook a study resembling that of Missouri’s study for rural multilane divided segments and four-leg signalized intersections in North Carolina. The calibration factors, obtained, were 0.97 and 0.49 respectively. In Oregon, Xie et al. (2011) calibrated the HSM’s SPF for rural divided multilane highway segments and
achieved a calibration factor of 0.78 indicating that the HSM’s SPF considerably over-predicted crash counts for Oregon’s conditions. Mehta and Lou (2013), not only adopted the HSM’s SPFs for rural divided segments but also developed a local SPF for Alabama’s conditions. The local SPF outperformed the borrowed one from the HSM. Brimely et al. (2012) borrowed the HSM’s SPF for predicting total crash counts in Utah’s rural two lane roads and achieved a calibration factor of 1.16. That is, the HSM’s SPF under-predicted crashes for Utah’s conditions by 16%. The study’s researchers also developed an own localized SPF that fitted their crash data better than the HSM’s SPF. In Regina, Saskatchewan, Canada, Young and Park (2012) borrowed the HSM’s SPF for signalized and unsignalized intersections. They also developed local SPFs and suggested the use of the local SPFs over those of the HSM. In Toronto, Persaud et al. (2002) estimated SPFs for both stop-controlled intersections and signalized intersections. The SPFs were calibrated to conditions of Vancouver and California via the HSM calibration method. The conclusion was that the calibrated SPFs under-predicted crashes by a large amount. However, the research team used cumulative residual (CURE) plots (Hauer and Bamfo 1997) and found that the predictions are not faulty for particular ranges of the AADT.

Calibrating the HSM’s SPFs was undertaken abroad North America as well. The HSM’s SPFs of fatal and injury crashes for urban roads were calibrated for conditions in Riyadh, Saudi Arabia and the calibration factor, computed, was 0.31 signifying that the HSM’s SPF over-predicted crashes by 69% (2015). The research team achieved more appropriate model fits by developing local SPFs with a variety of specifications. In addition, the HSM’s SPFs were tested to check whether they were applicable to the A-18 motorway in Messina-Catania, Italy (Cafiso et al. 2012; D’Agostino 2014). Calibration factors, achieved, implied that the HSM under-predicted crashes by a substantial amount. Other than the Italian studies, the HSM’s SPFs were calibrated for urban intersections in Fortaleza City, Brazil as well (Cunto et al. 2015). The authors of the study obtained a calibration factor of 0.98 for conditions of signalized intersections and 2.15 for conditions of unsignalized intersections. That is, the HSM’s SPFs almost accurately predicted crash counts at signalized intersections and under-predicted crash frequencies by 115% at unsignalized intersections.

Other than adopting and calibrating the HSM’s SPFs, Srinivasan et al. (2016) introduced a calibration technique, referred to as the calibration function in this paper. It is shown in Equation (6).

\[ N_{obsi} = \hat{a} \times \hat{N}_{SPFi} \]

(6)

Equation (6) is a NB model and the terms, \( \hat{a} \) and \( \hat{b} \), are coefficients estimated using the MLE method. It should be noted that if \( \hat{b} \) is not statistically significantly different from 1, Equation (6) reduces to that of the HSM calibration technique.

Generally, the adoption of the HSM’s SPFs by agencies is a topic of interest. The aim of this research is not to evaluate the transferability of the HSM’s SPFs to a particular jurisdiction but to develop SPFs for 3 states and determine whether the SPFs are transferable. The states are Florida, California and Washington. The SPFs are for rural divided multilane highway segments. Also, the crash category, analyzed, is that of total crashes, designated as KABCO in the HSM. The following stage of this research is the application of both the HSM’s calibration technique and the calibration function for calibrating the transferred SPFs to the 3 states. The data collection and statistics are described in the subsequent section.

DATA PREPARATION

The data consist of crash records, geometric characteristics and traffic characteristics. The Florida crash data are collected from the Crash Analysis Reporting System (CARS) database of the Florida Department of Transportation (FDOT). The geometric and traffic characteristics’ data are collected from the Roadway Characteristics Inventory (RCI) database which also belongs to the FDOT. On the other hand, all data from California and Washington are collected from the Highway Safety Information System (HSIS), a national database. Florida and Washington’s crash data records, collected, are those of the years 2009 to 2011 while those of California are those of 2009 to 2010. The 2011 crash data of California are not available in the HSIS database. The descriptive statistics of the data are shown in Table 1. The minimum segment length is 0.1 mi (0.161 km) to adhere to the HSM standards. The crash rates, which are crashes normalized by the vehicle kilometers of travel (VKT) per year, are depicted in Figure 1. As shown in the figure, the rates are not comparable.
### Table 1. Data Descriptive Statistics

<table>
<thead>
<tr>
<th>Variable</th>
<th>Frequency</th>
<th>Mean</th>
<th>Standard Deviation</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Florida (436 segments, 564.302 km, crash years: 2009-2011)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Segment Length (km)</td>
<td>-</td>
<td>1.294</td>
<td>2.348</td>
<td>0.161</td>
<td>29.094</td>
</tr>
<tr>
<td>AADT (vpd)</td>
<td>-</td>
<td>12,681.930</td>
<td>8,709.680</td>
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<td>49.500</td>
</tr>
<tr>
<td>Lane Width (m)</td>
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<td>3.660</td>
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<td>3.048</td>
<td>3.962</td>
</tr>
<tr>
<td>Shoulder Width (m)</td>
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<td>1.448</td>
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<tr>
<td>Median Width (m)</td>
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<td>12.323</td>
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<td><strong>California (1,153 segments, 958.042 km, crash years: 2009-2010)</strong></td>
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<td></td>
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</tr>
<tr>
<td>Segment Length (km)</td>
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<td>0.062</td>
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<td>Median Width (m)</td>
<td>-</td>
<td>13.015</td>
<td>9.217</td>
<td>1.524</td>
<td>30.175</td>
</tr>
<tr>
<td>Crashes</td>
<td>3,887</td>
<td>3.371</td>
<td>6.182</td>
<td>0</td>
<td>71</td>
</tr>
<tr>
<td><strong>Washington (292 segments, 183.472 km, crash years: 2009-2011)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Segment Length (km)</td>
<td>-</td>
<td>0.628</td>
<td>0.647</td>
<td>0.166</td>
<td>3.819</td>
</tr>
<tr>
<td>AADT (vpd)</td>
<td>-</td>
<td>14,914.530</td>
<td>7,578.160</td>
<td>3.947</td>
<td>42.310</td>
</tr>
<tr>
<td>Lane Width (m)</td>
<td>-</td>
<td>3.663</td>
<td>0.040</td>
<td>3.658</td>
<td>3.962</td>
</tr>
<tr>
<td>Shoulder Width (m)</td>
<td>-</td>
<td>2.957</td>
<td>0.250</td>
<td>1.372</td>
<td>3.200</td>
</tr>
<tr>
<td>Median Width (m)</td>
<td>-</td>
<td>15.275</td>
<td>8.427</td>
<td>1.219</td>
<td>54.864</td>
</tr>
<tr>
<td>Crashes</td>
<td>893</td>
<td>3.058</td>
<td>3.794</td>
<td>0</td>
<td>24</td>
</tr>
</tbody>
</table>
RESEARCH METHODOLOGY

The methods used in this research to develop and assess the transferability of SPFs of the 3 states are described. In addition, outlining of the calibration procedures are presented.

Safety Performance Function Development

An SPF, for roadway segment, \(i\), is developed for KABCO crashes for each state. The functional form of the SPFs and overdispersions are given by Equations (7) and (8) respectively.

\[
\hat{\beta}_{SPFi} = \exp\left[\hat{A} + \hat{B} \times \ln(AADT_i) + \hat{D} \times (S_{wi}) + \hat{E} \times (M_{wi}) + \ln(L_i \times y_r)\right]
\]

(7)

\[
\hat{k}_i = \frac{1}{\exp\left[\hat{C} + \ln(L_i \times y_r)\right]}
\]

(8)

In Equation (7), \(S_{wi}\) and \(M_{wi}\) are the segment’s right shoulder width and median width respectively. Also, \(y_r\) is the number of years of which the crash data are collected. As previously mentioned, Florida and Washington’s crash data are those of 3 crash years (2009-2011) whereas those of California are of 2 years (2009-2010). The number of years term is to account for the fact that the SPF predicts the crash counts per year. The SPF specification and dispersion parameter are similar to those of the HSM except that the shoulder width and median width parameters are included to compensate for the CMFs. The SPFs are run using the NLMIXED procedure in base SAS, a statistical software package, and the statistically insignificant parameter at the 95th percentile confidence interval with the largest p-value is removed. Once the variable is removed, the SPF is rerun and the variable removal process is repeated until all parameters in the resulting SPF are statistically significant. The process of removing insignificant variables one by one is known as backward elimination.

Safety Performance Function Transferability Assessment

Once the SPFs are developed, they are applied to the data of the other states and the transferability of the SPFs are measured using a transfer index, \(TI\) (Hadayeghi et al. 2006; Sikder et al. 2014), defined as follows.

\[
TI = \frac{LL(\beta_{jreference}) - LL(\beta_{j})}{LL(\beta_{j}) - LL(\beta_{jreference})}
\]

(9)

The log-likelihood of the transferred SPF, which is a measure of how accurate it is predicting crashes at the destination jurisdiction it is being transferred to is \(LL(\beta_{j})\). The log-likelihood of the null SPF of the destination jurisdiction, which is an SPF with only the terms, \(\hat{A}\) and \(\ln(L_i \times y_r)\), is \(LL(\beta_{jreference})\). The log-likelihood of the SPF of the destination jurisdiction is \(LL(\beta_{j})\). In essence, the index is used to compare the performance of the transferred SPF with that of the destination jurisdiction’s null SPF at predicting crash counts for conditions in the destination jurisdiction. An index of 1
indicates that the transferred SPF performs as well as that of the destination jurisdiction’s SPF. An index of 0 indicates that the performance of both the transferred SPF and the destination jurisdiction’s null SPF are the same. On the other hand, a negative index implies that the transferred SPF underperforms the destination jurisdiction’s SPF. Hence, an index between 0 and 1 is desirable.

Safety Performance Function Calibration

It is not necessary that an SPF is transferable to conditions of a particular jurisdiction elsewhere. Hence, calibration is required. Both the HSM calibration technique and Srinivasan et al.’s (2016), also known as the calibration function, are conducted to calibrate the transferred SPFs in this research. The goodness of fit (GOF) measure used for assessing the accuracy of the predictions of the SPFs after calibration is the mean absolute deviation (MAD). It is defined as follows.

\[
MAD = \frac{\sum_{i=1}^{n} |N_{SPF_i} - N_{obsi}|}{n}
\]  

(10)

Bootstrapping is conducted as a further step. From the destination jurisdiction’s data, 100 samples, of sizes \(n\), are collected with replacement. Consequently, this yields to the computation of 100 calibration factors and thus 100 MADs of which mean and standard deviation can be obtained. The mean and standard deviation are used to calculate the 95\(^{th}\) percentile confidence limits of the MAD. Bootstrapping is conducted using the package “boot” (Canty and Ripley, 2017) of the statistical software package R.

EMPIRICAL ANALYSIS

The SPFs are developed successfully. Their results are presented in Table 2. The predictors, shoulder width and median width, are input in the SPFs in meters while the segment length is input in kilometers. On the other hand, in the SPFs provided by the HSM, inputs are required to be in US customary units. The SPFs, of which results are presented in Table 2, are applied to each state and the transfer indices are computed. The transfer index results are shown in Table 3.

Table 2. Safety Performance Function Results

<table>
<thead>
<tr>
<th>State</th>
<th>Parameter</th>
<th>Regression Coefficient with P-Value (in Parenthesis)</th>
<th>SPF</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Constant</td>
<td>Ln(AADT)</td>
<td></td>
</tr>
<tr>
<td>Florida</td>
<td></td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>-6.701</td>
<td>0.720</td>
<td>-0.091</td>
</tr>
<tr>
<td></td>
<td>( &lt; 0.001)</td>
<td>( &lt; 0.001)</td>
<td>(&lt; 0.001)</td>
</tr>
<tr>
<td></td>
<td>Shoulder Width</td>
<td>-0.323 (0.006)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Median Width</td>
<td>-0.323 (0.006)</td>
<td></td>
</tr>
<tr>
<td>California</td>
<td></td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>-9.904</td>
<td>1.120</td>
<td>-3.387</td>
</tr>
<tr>
<td></td>
<td>( &lt; 0.001)</td>
<td>( &lt; 0.001)</td>
<td>(&lt; 0.001)</td>
</tr>
<tr>
<td></td>
<td>Shoulder Width</td>
<td>-0.207 (0.001)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Median Width</td>
<td>-0.207 (0.001)</td>
<td></td>
</tr>
<tr>
<td>Washington</td>
<td></td>
<td></td>
<td>0.36</td>
</tr>
<tr>
<td></td>
<td>-3.893</td>
<td>0.480</td>
<td>10.087</td>
</tr>
<tr>
<td></td>
<td>( &lt; 0.001)</td>
<td>( &lt; 0.001)</td>
<td>(&lt; 0.001)</td>
</tr>
<tr>
<td></td>
<td>Shoulder Width</td>
<td>-0.012 (0.048)</td>
<td>0.228</td>
</tr>
<tr>
<td></td>
<td>Median Width</td>
<td>-0.012 (0.048)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SPF</td>
<td>SPF</td>
<td>SPF</td>
</tr>
<tr>
<td></td>
<td>Florida</td>
<td>SPF</td>
<td>SPF</td>
</tr>
<tr>
<td></td>
<td>California</td>
<td>SPF</td>
<td>SPF</td>
</tr>
<tr>
<td></td>
<td>Washington</td>
<td>SPF</td>
<td>SPF</td>
</tr>
</tbody>
</table>

Note: * statistically insignificant variable at 95\(^{th}\) percentile confidence interval removed

It should be noted the according to the SPF results, shown in Table 2, larger AADTs, which is a measure of exposure to crashes, translate to more crashes as indicated by the estimated coefficients’ signs. Yet, the relationship between crash counts and traffic volumes are not linear as shown in the table. Also, wider shoulders and medians translate to safer roads possibly by giving adequate room for drivers veering off the road to recover.

Table 3. Transfer Index Results

<table>
<thead>
<tr>
<th>SPF</th>
<th>Application Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Florida</td>
<td>Florida</td>
</tr>
<tr>
<td>California</td>
<td>-3.387</td>
</tr>
<tr>
<td>Washington</td>
<td>-10.087</td>
</tr>
</tbody>
</table>

As shown in Table 3, not a single state’s SPF is transferable to another state’s data and vice versa. Explanations regarding the factors that hinder SPF transferability can be posited. Differences in crash reporting thresholds, which are the minimum damage to property required for a crash to be reported, socio-demographic characteristics, weather effects and topographic characteristics may impede SPFs from being transferable. In addition, socio-demographics heavily influence the driver’s behavior since more diverse populations introduce a variety of driving habits. Also, drivers, unfamiliar with the roads, especially tourists, are more susceptible to be involved in crashes.
Florida’s terrain is level and is known for its dense woodlands. It characterized by rainy summers and autumns. The annual rainfall intensity ranges from 100 cm to 155 cm (Florida 2017). Furthermore, Florida has a considerable population of elderly, especially during the winter, and a variety of ethnic groups mainly in large urban areas. Furthermore, the crash reporting threshold is $500 worth of damage to property, injury or death (Accident Reporting 2017).

Unlike Florida, California has hilly and mountainous terrains, forests in the north and deserts near the eastern border (California 2017). California experiences much less rainfall than Florida except in the north where, on average, 50 cm of rain is experienced annually. Snow activity is present in the mountainous regions (California 2017) but the data of this research do not cover the snowy regions of California. Tourism is lively in California as it is in Florida (Visit California 2017). Additionally, the reporting threshold is either an injury, fatality or $750 worth of property damage prior to January, 2017 (Accident Reporting 2017).

Washington is characterized by its Cascade Range mountains, Olympia mountains and the flat Puget Sound region. Level and hilly terrain are also present (Washington 2017). Furthermore, Washington is also known for its heavy rainfall intensity but not its tourism, unlike Florida and California. Snowfall is active, especially within the mountainous areas. In addition, injury crashes, fatal crashes or any crashes with at least $700 worth of property damage ought to be reported (Accident Reporting 2017).

Calibration of Safety Performance Functions

The SPFs are calibrated to each jurisdiction once by the HSM’s technique and once by the calibration function. Results of the MADs with confidence limits, computed by bootstrapping, are presented in Tables 4 and 5.

Table 4. Highway Safety Manual Calibration Factors with 95th Percentile Confidence Limits

<table>
<thead>
<tr>
<th>SPF</th>
<th>Application Data</th>
<th>Florida</th>
<th>California</th>
<th>Washington</th>
</tr>
</thead>
<tbody>
<tr>
<td>FL</td>
<td>(0.880, 1.123)*</td>
<td>(2.287, 2.705)</td>
<td>(3.288, 3.950)</td>
<td></td>
</tr>
<tr>
<td>CA</td>
<td>(0.459, 0.599)</td>
<td>(0.945, 1.113)*</td>
<td>(1.492, 1.824)</td>
<td></td>
</tr>
<tr>
<td>WA</td>
<td>(0.365, 0.476)</td>
<td>(0.955, 1.137)*</td>
<td>(0.912, 1.090)*</td>
<td></td>
</tr>
</tbody>
</table>

Note* Calibration factor not statistically significantly different from 1

Table 5. Calibration Methods' MAD Results with 95th Percentile Confidence Limits

<table>
<thead>
<tr>
<th>SPF</th>
<th>Calibration Method</th>
<th>Application Data</th>
<th>Florida</th>
<th>California</th>
<th>Washington</th>
</tr>
</thead>
<tbody>
<tr>
<td>FL</td>
<td>HSM</td>
<td>(1.437, 2.029)</td>
<td>(2.089, 2.627)</td>
<td>(1.556, 2.010)</td>
<td></td>
</tr>
<tr>
<td>CA</td>
<td>HSM</td>
<td>(1.483, 2.104)</td>
<td>(1.974, 2.483)</td>
<td>(1.670, 2.194)</td>
<td></td>
</tr>
<tr>
<td>WA</td>
<td>HSM</td>
<td>(1.453, 2.082)</td>
<td>(2.232, 2.807)</td>
<td>(1.513, 1.953)</td>
<td></td>
</tr>
<tr>
<td>FL</td>
<td>Calibration Function</td>
<td>(1.417, 2.071)</td>
<td>(2.061, 2.656)</td>
<td>(1.523, 2.000)</td>
<td></td>
</tr>
<tr>
<td>CA</td>
<td>Calibration Function</td>
<td>(1.452, 2.118)</td>
<td>(1.965, 2.480)</td>
<td>(1.577, 2.084)</td>
<td></td>
</tr>
<tr>
<td>WA</td>
<td>Calibration Function</td>
<td>(1.465, 2.107)</td>
<td>(2.198, 2.861)</td>
<td>(1.486, 1.959)</td>
<td></td>
</tr>
</tbody>
</table>

The results of Table 4 are consistent with those of the transfer indices since they both indicate that not a single pair of states have mutually transferable SPFs. Florida’s SPF, by a large degree, under-predicts crashes in California and Washington. California’s SPF over-predicts crashes in Florida and under-predicts crashes in Washington. Washington’s SPF over-predicts crashes in Florida. Furthermore, the calibration factor of Washington’s SPF applied to California is not statistically significantly different from 1 even though California’s SPF under-predicts crashes in Washington. When it comes to comparison of the calibration techniques, it can be inferred from Table 5 that there is no significant difference in MADs of transferred SPFs calibrated by the HSM technique and of those calibrated using the calibration functions.

CONCLUSIONS

This research is aimed at investigating the transferability of SPFs of total crashes of Florida, California and Washington states. The SPFs’ functional forms are similar to those of the HSM except that the shoulder width and median width parameters are included in addition to the number of crash years since the data are not limited to the HSM’s defined base conditions. As per the transfer index results, which are used to assess SPF transferability, the states’ SPFs are not transferable to each jurisdiction. The HSM calibration factor method is applied to calibrate the SPFs to each state as
The results of the HSM calibration factors are consistent with those of the transfer index results. The deterrence of the transferability of SPFs may explained by differences in weather conditions, topographic conditions, socio-demographic characteristics and crash reporting thresholds. Also, the recent calibration technique, proposed by Srinivasan et al (2016), is applied and its performance is demonstrated to be not superior to that of the HSM method. Research into the development of a more accurate calibration technique is warranted.

This research offers several practical implications. One is that jurisdictional agencies may adopt SPFs from elsewhere and thus save the costs of data collection, management, preparation and processing if the borrowed SPF is demonstrated to be transferable to the conditions of interest. Another implication is that the finding that a more robust calibration technique is worthy of development is advantageous. That is because calibration techniques are instrumental for practitioners to apply SPFs to specific localities without implementing computationally intensive procedures.

ACKNOWLEDGMENTS

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REFERENCES


Identification and prioritization of accident black-spots in Ernakulam district

ABSTRACT:
Roads in Kerala face problems like congestion, air pollution and an alarming number of traffic accidents, which are increasing with the growing motorization rate. The major objectives of the study are to identify the road accident black-spots and to prioritize them for implementation of rectification measures. Accident data for the past three years (2013-2015) were collected, compiled and refined to assess existing accident scenario and trend in Ernakulam district. All accident spots involving two or more road accidents involving fatal or grievous accidents were identified and GPS co-ordinates of 12,241 accident locations were established. All the selected accidents were attributed with accident characteristics and mapped on GIS Software. A total of 412 high risk locations were identified in Ernakulam district based on MoRTH's criteria of black-spots using analytical tools in GIS software. Hazardous locations were evaluated and ranked based on Accident Severity index (ASI), which depends on the severity and frequency of accidents. A total of 16 first order black spots, have been identified based on ASI value, which should be rectified immediately on priority basis. 16 second order black spots and 35 third order black spots have been identified, which should be rectified on second and third order priority basis respectively. Heat maps were generated in GIS Software, which uses the Getis-Ord Gi* algorithm. Collision type characteristics of road accidents in accident black spots were identified.

KEYWORDS:
Accident, black-spots, crash spots, Accident Severity Index, GIS, Ernakulam
Identification and prioritization of accident black-spots in Ernakulam district

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ebin.natpac@gmail.com, bgsreedevi@yahoo.com

1 INTRODUCTION

In 2015, Kerala has witnessed 39,014 road accidents resulting in 4,196 fatalities, 29,096 persons with grievous injuries and 14,639 persons with minor injuries. In Kerala, 116.6 persons were injured in road accidents per lakh population, which is nearly three times the national average and the highest in the country. During the year 2015 the state of Kerala ranked the highest (29,096) in the total number of persons grievously injured in the country.

Roads in Kerala face problems like congestion, air pollution and an alarming number of traffic accidents, which are increasing with the growing motorization rate. In order to operate roads safely, it is necessary to monitor accident occurrence and to assess the scope for remedial treatment to reduce accident numbers and severity. Safety of existing roads which are fully operational, may be improved through several types of procedures in Road Safety Engineering: Black Spot Management, Network Safety Management, Road Safety Audit and Road Safety Inspections.

Black Spot Management (BSM), which is a reactive approach within the discipline of traffic safety, involves scientific analysis of accident data, identifying the nature and cause of accidents and designing appropriate engineering interventions leading to prevention of such road accidents in future. Black spots are defined as locations noted for high concentration of road accidents involving death and injury than other similar locations as a result of local risk factors. Identification, analysis and treatment of these black spots are widely regarded as one of the most effective approaches in preventing road crashes.

A well-organized, comprehensive, and accessible data collection system is crucial for identifying the individual black spots. There is an urgent need to identify the black spots and prioritize them so that scarce resources can be utilized more judiciously for road safety measures. At the instance of Kerala Road Safety Authority, NATPAC has conducted study on identification and prioritization of accident black-spots in Ernakulam district, which has reported increased number of accidents.

2 SCOPE AND OBJECTIVES

The purpose of the study is to identify the road accident black-spots and prioritize them for implementation of rectification measures in Ernakulam district. The following tasks/services were carried out as part of the study:

- Collection and refinement of road accident data from Police department for a period of three years;
- Assessment of existing road accident scenario and trend in Ernakulam District;
- Identification of black-spot locations on the road stretches in Ernakulam District;
- Prioritization of hazardous locations for rectification through scientific methods on Geographic Information System (GIS);
- Visualization of accident spots on maps to highlight vulnerable locations in the district;
- Identification of characteristics of identified accident black-spots in the district.

3 BRIEF PROFILE OF THE STUDY AREA

Ernakulam is situated in the central part of that state, which is known as the commercial capital of Kerala. The district headquarters is located at Kakkanad, a suburb of Kochi city. The district is famous for its ancient temples, churches, and mosques. The district includes the largest metropolitan region of the state, Greater Cochin. Ernakulam district is the highest revenue-yielding district in the state. Ernakulam is the third most populous district in Kerala, after Malappuram and Thiruvananthapuram. In 2011, Ernakulam had population of 3,282,388 of which male and female were 1,619,557 and 1,662,831 respectively. The decadal population growth rate of Ernakulam district during 2001-2011 is 5.69% and 1991-2001 is 10.24%. Average literacy rate of Ernakulam district is 95.89 percent of which male and
female literacy was 97.36 and 94.46 percent respectively. Map showing Ernakulam district in Kerala is shown in Figure 1.

Ernakulam district is well served by four dominant modes of transport viz. road, rail, water and air. Of these, road network has a wider presence throughout the district. Inland navigation plays a significant role in ferrying people from islands and backwater region to the mainland and is confined to western parts of the city.

Ernakulam district has extensive road network, maintained by PWD, NHAI and local bodies. There are five National Highways in the district connecting various locations namely NH 85, NH 544, NH 66, NH 966 A and NH 966 B. Ernakulam district is connected by State Highways and Major District Roads, which are maintained by Kerala State PWD. The district has 325.206 km of State Highways and 2815.555 km of Major District Roads under the jurisdiction PWD Ernakulam Roads Division and PWD Muvattupuzha Roads Division. Ernakulam district is served by public transport system consisting of both State owned Kerala State Road Transport Corporation (KSRTC) and private buses. The road network of Ernakulam district is given in Figure 2.
Kochi Metro is a mass rapid transit system under construction for Kochi city in the district of Ernakulam. The 25.65 km metro line runs from Aluva to Petta and includes 22 stations. Railways are the cheapest and the most convenient mode of journey and opted by majority of the population as it caters mainly to the needs of inter-city passenger and goods traffic. Ernakulam district has 17 railway stations in the district of which the most prominent ones being Ernakulam town (Ernakulam North) and Ernakulam Junction (Ernakulam South).

Ernakulam has a good network of inland waterway system consisting of backwaters, canals, lagoons and estuaries. National Waterway No.3 connecting Kollam and Kottappuram pass through the region. The State Water Transport Department (SWTD), Kerala Shipping and Inland Navigation Corporation (KSINC) and private operators are providing passenger and cargo boat services to the adjoining islands and industrial centers located in this region.

Cochin Port is a major port on the Laccadive Sea – Indian Ocean sea route and is one of the largest ports in India. The port lies on two islands namely Willingdon Island and Vallarpadam. The International Container Transshipment Terminal (ICTT), part of the Cochin Port located at Vallarpadam, is the largest container transshipment facility in India.

Cochin International Airport is located at Nedumbassery, which is nearly 28 km from Kochi City, caters to the needs of domestic and international passengers of Ernakulam district and surrounding regions. International airport handled 46,41,127 international passengers and 31,29,658 domestic passengers during the year 2015-16.

4 ACCIDENT SCENARIO AND TREND

In Ernakulam District, more than 17,190 accidents occurred resulting in 20,595 victims during three years (2013-2015) as per accident raw data obtained from SCRB and police stations. During three calendar years, 1484 people died, 12,150 people were grievously injured and 6961 people suffered minor injuries in road accidents in Ernakulam district. 41% of accidents during three years occurred within city limits, where as 59% of accidents in rural police station limits. 75% of road accidents in Ernakulam district occur during daytime. Share of accidents in Ernakulam district during three years according to accident type is given in Figure 3 and share of road accident victims in Ernakulam district is given in Figure 4.

Figure 3: Share of accidents according to type of accidents in Ernakulam district

Figure 4: Share of victims according to severity in Ernakulam district
In Ernakulam district, 26% of accidents are of pedestrian hit collision type followed by Rear End (20%), Side Hit (19%) and Head On (17%) collision types. Passenger Transport vehicles were involved in 76%, Non Motorized Transport in 14% and Goods Vehicle in 8% of total road user type involved in accidents. Among the accidents during three calendar years in Ernakulam district, 44% of road users involved were two wheeler, followed by Car/Jeep (17%) and Pedestrians (11%). Buses and Passenger Auto rickshaw were involved to 8% and 7% respectively. Share of road accidents according to type of collision is given in Figure 5. Share of type of Motorized Vehicle/Non Motorist Transport involved in Ernakulam district is given in Figure 6.

![Figure 5: Share of accidents according to collision type in Ernakulam district](image1)

![Figure 6: Type of Motorized Vehicle/Non Motorist Transport Involved in Ernakulam district](image2)

It is important to compute accident rate which reflect accident involvement by different parameters such as population, area, length of road, number of registered vehicles etc. These rates provide means of comparing the relative safety of different types of roads, types of vehicles and road users. Crash indicators of Ernakulam district are given in Table 1.

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Indicators</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Fatalities per 100 accidents</td>
<td>8.64</td>
</tr>
<tr>
<td>2</td>
<td>Persons Injured per 100 accidents</td>
<td>111.33</td>
</tr>
<tr>
<td>3</td>
<td>Accidents per kilometre on NH</td>
<td>7.65</td>
</tr>
<tr>
<td>4</td>
<td>Accidents per kilometre on SH</td>
<td>2.79</td>
</tr>
<tr>
<td>5</td>
<td>Accidents per kilometre on OR</td>
<td>1.14</td>
</tr>
<tr>
<td>6</td>
<td>Accidents per lakh population</td>
<td>170.66</td>
</tr>
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<td>7</td>
<td>Fatalities per lakh population</td>
<td>14.74</td>
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<tr>
<td>8</td>
<td>Injuries per lakh population</td>
<td>189.73</td>
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<td>9</td>
<td>Accidents per 10,000 registered vehicles</td>
<td>35.36</td>
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<tr>
<td>10</td>
<td>Fatalities per 10,000 registered vehicles</td>
<td>3.06</td>
</tr>
<tr>
<td>11</td>
<td>Persons Injured per 10,000 registered vehicles</td>
<td>173.60</td>
</tr>
<tr>
<td>12</td>
<td>Two wheelers involved per 100 accidents</td>
<td>77.20</td>
</tr>
</tbody>
</table>
The number of grievous accidents shows an increasing trend in the district during three years, whereas the number of fatal and minor accidents shows neither increasing nor diminishing trend. During the years from 2013 to 2015, number of grievously injured victims shows an upward trend, with numbers reaching 4300 victims. Trend of accident severity rate/fatality rate i.e., number of persons killed per 100 accidents in Ernakulam district remains constant. But the trend of persons getting injured for every 100 road accidents in the district witness an upward trend (around 2% growth) during the period 2013 to 2015. Severity trend based on type of accidents and based on victims in Ernakulam district are shown in Figure 7 and Figure 8 respectively. Figure 9 shows the trend of accident severity rate in Ernakulam district.

<table>
<thead>
<tr>
<th>13</th>
<th>Cars involved per 100 accidents</th>
<th>30.60</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>Pedestrians involved per 100 accidents</td>
<td>23.49</td>
</tr>
</tbody>
</table>

In Ernakulam district, trend of accident rate per kilometre on National Highways, State Highways and Other Roads show an increasing trend during three years (2013 - 2015). Average Annual Growth Rate of accidents per km on
NH, SH and OR is 3.4%, 3.2% and 3.6% respectively. In Ernakulam district, trend of accident risk of two wheelers is increasing at an alarming level during the period 2013-2015. Accident risk of Cars and non-motorized Transport in the district show a slightly declining trend. Trend of accidents per km rate on National Highways, State Highways and Major District Roads in Ernakulam district is given in Figure 10. Figure 11 shows the trend of accident risk of two wheelers, cars and NMT users in Ernakulam district.

![Figure 10: Accidents per km trend according to road type in Ernakulam district](image1)

![Figure 11: Trend of accident risk of two wheelers, cars and NMT users in Ernakulam district](image2)

4 MAPPING OF ACCIDENT SPOTS

Accident spots of three calendar years (2013-2015) were identified with the help of SCRB data, additional data from local police station and police personnel. All accident spots involving two or more road accidents involving fatal or grievous accidents were identified and GPS co-ordinates were established using Hand Held GPS device. GPS co-ordinates of 12,241 accident locations with two or more fatal/grievous accidents were established during the field survey. All the individual accidents (12,241 Nos) were attributed with accident characteristics for further analysis. Major attributes comprising of accident characteristics for mapping of accidents are listed in Table 2.

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Major Attributes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Crime Number</td>
</tr>
<tr>
<td>2</td>
<td>GPS co-ordinates</td>
</tr>
<tr>
<td>3</td>
<td>Name of Police Station</td>
</tr>
<tr>
<td>4</td>
<td>Accident Date &amp; Time</td>
</tr>
<tr>
<td>5</td>
<td>Name of road, type of road, location and landmark of accident spot</td>
</tr>
<tr>
<td>6</td>
<td>Type of accident (Fatal/Grievous/Minor/Non Injury)</td>
</tr>
<tr>
<td>7</td>
<td>Number of victims (Fatalities, Grievous Injuries, Minor Injuries)</td>
</tr>
<tr>
<td>8</td>
<td>Type of Collision</td>
</tr>
<tr>
<td>9</td>
<td>Accident Cause according to FIR</td>
</tr>
<tr>
<td>10</td>
<td>Type of Motorized Vehicles/Non Motorized Transport involved</td>
</tr>
</tbody>
</table>
All the selected accidents along with the attributes were digitized and mapped on ArcGIS Software for further analysis and identification of black spots. Map showing identified fatal and grievous accident in Ernakulam district is shown in Figure 12.

Figure 12. Map showing fatal and grievous accident spots in Ernakulam district

5 IDENTIFICATION OF BLACK SPOTS

Locations where the accidents repeatedly take place or tend to cluster together are commonly termed as “Accident Black Spots” or “Crash Hot Spots”. Locations are generally classified as black spots after an assessment of the level of risk and the likelihood of a crash occurring at a location. Ministry of Road Transport & Highways\(^1\) (MoRTH)'s guideline of road accident black spot has been adopted for identification of high risk areas. As per MoRTH, black spot is a road stretch of about 500m in length in which either 5 road accidents (involving fatalities/grievous injuries) took place during last three calendar years or 10 fatalities took place during last three calendar years.

Black spots are identified from the spatial interaction existing between contiguous accident locations. The use of Geographical Information Systems and point pattern techniques, which can identify such zones, is deployed in this study. GIS-based accident information systems provide a platform to conduct spatial analysis of the accident data which are almost impossible by using a non-spatial database. The road network and selected crash data were loaded in the ArcGIS software for identifying black spots. Each data points represent individual accidents of those locations which had recorded two or more road accidents involving fatal or grievous accidents during last three calendar years in Ernakulam district.

A total of 412 high risk locations were identified in Ernakulam district based on MoRTH's criteria of black spots. Map showing the high risk areas based on MoRTH's criteria is shown in Figure 13.

Figure 13. Map showing high risk areas based on MoRTH's criteria
5 PRIORITIZATION OF BLACK SPOTS

It is necessary to prioritise between locations and safety measures in order to utilize the scarce funds more effectively. For prioritization of crash hot spots, National Highway Authority of India's (NHAI) methodology has been adopted. According to NHAI, hazardous locations are evaluated and ranked based on Accident Severity index (ASI). ASI depends on the severity and frequency of accidents. For estimation of ASI, the weight age to fatal accident will be assigned as 7 and to grievous injury accident as 3.

\[
\text{Accident Severity Index, } ASI = 7F + 3GI
\]

Where,
\[
F = \text{Number of Fatal Accidents};
\]
\[
GI = \text{Number of Grievous Accidents}.
\]

Accident Severity Index (ASI) values of hazard prone stretches based on MoRTH's criteria were computed in ArcGIS software. Map showing the ASI values of the accident prone stretches is shown in Figure 14.

![Figure 14. ASI values of the accident prone stretches in Ernakulam district](image)

As per NHAI, hazardous spots with Accidents Severity Index (ASI) more than threshold value will be treated as Black spots. Threshold value was computed in ArcGIS Software and found to be 178.03, which was based on NHAI's criteria. ASI values used to prioritize black spots are listed in Table 3.

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Order of black spots</th>
<th>Computation Formula</th>
<th>Threshold value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>First order black spots</td>
<td>Average Severity + 1.5*Standard Deviation</td>
<td>178.03</td>
</tr>
<tr>
<td>2</td>
<td>Second order black spots</td>
<td>Average Severity + Standard Deviation</td>
<td>141.74</td>
</tr>
<tr>
<td>3</td>
<td>Third order black spots</td>
<td>Average Severity + 0.5*Standard Deviation</td>
<td>105.44</td>
</tr>
</tbody>
</table>

All hazardous stretches whose ASI value above threshold value of 178.03, were considered as first order black spots. The zones that are classified under first order black spots are highly accident prone, which should be given highest priority for implementing suitable counter measures. A total of 16 first order black spots have been identified, which should be rectified immediately on priority basis.

ASI values of hazardous spots, which lies between 178.03 and 141.74, were considered as second order black spots. A total of 16 second order black spots have been identified, which should be rectified on second order priority basis. ASI values in between 141.74 and 105.44 were considered as third order black spots. A total of 35 third order black spots have been identified, which should be rectified on third order priority basis.
6 GEOSPATIAL REPRESENTATION OF ACCIDENT SPOTS

Hazardous locations were geospatially visualized on Geographic Information System (GIS) Platform by representing the accidents on the map. Heat maps, which display high frequency crash locations of different types, were generated in ArcGIS Software using Spatial Statistics Hot Spot Analysis tool, which uses the Getis-Ord Gi* algorithm. Detailed and focused measures can be adopted on the road stretches, which can significantly reduce the accident risks in the district.

The Hot Spot Analysis tool calculates the Getis-Ord Gi* statistic for each feature in a dataset. The resultant Z score tells you where features with either high or low values cluster spatially. This tool works by looking at each feature within the context of neighboring features. A feature with a high value is interesting, but may not be a statistically significant hot spot. To be a statistically significant hot spot, a feature will have a high value and be surrounded by other features with high values as well. The local sum for a feature and its neighbors is compared proportionally to the sum of all features; when the local sum is much different than the expected local sum, and that difference is too large to be the result of random chance, a statistically significant Z score results.

The Gi* statistic returned for each feature in the dataset is a Z score. For statistically significant positive Z scores, the larger the Z score is, the more intense the clustering of high values (hot spot). For statistically significant negative Z scores, the smaller the Z score is, the more intense the clustering of low values (cold spot). The Gi* statistics is a z - score and hence further calculations are not required.

The Getis-Ord local statistic is given as:

\[
G_{ij}^* = \frac{\sum_{j=1}^{n} w_{ij} x_j - \overline{X} \sum_{j=1}^{n} w_{ij}}{\sqrt{\frac{n \sum_{j=1}^{n} w_{ij}^2 - (\sum_{j=1}^{n} w_{ij})^2}{n-1}}}
\]

where \( x_j \) is the attribute value for feature \( j \), \( w_{ij} \) is the spatial weight between feature \( i \) and \( j \), \( n \) is equal to the total number of features and:

\[
\overline{X} = \frac{\sum_{j=1}^{n} x_j}{n}
\]

\[
S = \sqrt{\frac{\sum_{j=1}^{n} x_j^2}{n} - \overline{X}^2}
\]

Heat map has been prepared with the selected accident spots for identification of black spots in Ernakulam district, which is given in Figure 16. Heat map prepared based on type of collisions for selected accident spots in Ernakulam district, were also prepared.
7 CHARACTERISTICS OF BLACK SPOTS

Accident black spots have some common deficiency with respect to other location, which can be identified from type of collision, road and traffic characteristics. Characteristics of road accidents in accident black spots give an insight to the type of treatment required at the particular black spot stretch. A total of 67 locations have been identified in Ernakulam as priority black spots in Ernakulam district. Collision type characteristics of these black spots will aid appropriate selection of road safety engineering measures.

During three years (2013-2015), around 25% of total accidents in Ernakulam district occurred at the identified black spots in Ernakulam district. 28% of fatal accidents and 25% of grievous injury accidents happened on these accident black spots. Nearly 20% of road accidents victims in Ernakulam district happened at these identified black spot stretches. 28% of fatalities and 24% of grievous injuries occurred at these crash hot spot locations. 15% of road users involved in accidents in Ernakulam district occurred at these black spot locations. 21% of goods vehicles, 20% of non motorized transport and 17% of passenger vehicles involved in road crashes happened on these black spot stretches in the district. 24% of container/tanker lorries, 23% each of mini bus and truck, 21% of pedestrians, 20% of mini lorry/GOODS van, 18% each of MAT, Two Wheeler and cars/jeep involved in crashes in the district occurred on these identified black spots. Among Rear hit and Pedestrian hit collisions in Ernakulam district, 22% each of both collision types occurred at these identified black spots. 21% of overturn collisions and 18% of side hit collisions occurred at these locations.

At Karukutty, which has recorded highest number of road accidents in Ernakulam district, major share of accidents (74%) occurred during daytime. Nearly half of the collisions (48%) were of the type ‘Hit Pedestrian’ collisions followed by 19% ‘Side Hit’ collisions and 14% ‘Rear End’ type collisions. Among vehicles, 37% of vehicles involved in accidents were car/jeep followed by two wheelers (27%) and heavy goods vehicle (16%).

8 CONCLUSIONS

The study was conducted by a team of interdisciplinary staff of NATPAC consisting of Traffic Engineers, Road Safety Engineers, GIS Specialist, Transport Analyst and Road Safety consultants for identification and prioritization of accident black spots in Ernakulam district. A total of 412 high risk locations has been identified in Ernakulam district. Of these 412 high risk locations, 67 locations has been identified in Ernakulam as priority black spots in Ernakulam district. These identified black spots were prioritized into first order, second order and third order black spots for rectification.

Deaths and injuries caused by road accidents are eminently preventable. The combination of 5-E’s of road safety (Engineering, Education, Enforcement, Emergency, Evaluation) activities will reduce road trauma. Effective coordination of road safety efforts across multiple sectors and stakeholders is critical for treatment of black spots.
9 ACKNOWLEDGEMENTS

The Study Team expresses its deep sense of gratitude to Transport Commissioner and Kerala Road Safety Authority for extending financial support to NATPAC and in showing keen interest in the study. We also express our gratitude to Joint Transport Commissioners, other senior officials of Transport Department for rendering valuable service for the conduct of the study.

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The field work, analysis and preparation of report were organized and rendered by the staff of NATPAC is gratefully acknowledged.

10 CITATIONS AND REFERENCES

CITATIONS


2. National Highway Authority of India (NHAI), Terms of Reference for “Appointment of Safety Consultants for PPP projects on DBFO basis”

REFERENCES


PAPER TITLE: Safety Effectiveness Evaluation of Adding Left-turn Lanes at Signalized Intersections: Fixed and Random Effect Negative Binomial Models

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KEYWORDS:
Four-leg Signalized Intersection, SPF, Panel Data, Random Effect, Driver’s Characteristics

ABSTRACT:
The main objective of this study is to estimate the safety effectiveness of left-turn lanes at four-leg signalized intersections incorporating time trends into consideration. Standard crash models using longer time frames and aggregated data do not consider time-varying effects on crash frequency, and can introduce error due to unobserved heterogeneity. This study compares between traditional Negative Binomial (NB) models which account for the overdispersion of data and Random Effect Negative Binomial (RENB) models which consider unobserved random effects of the sites due to their heterogeneity. The RENB model outperformed NB model; therefore, RENB models were chosen to explain the effects of the confounding factors on crash counts. The objective of this study was achieved by developing site-specific Safety Performance Functions (SPF) for 174 four-leg signalized intersections using panel data from the years 2005 through 2014. Wyoming crash data including traffic, weather and driver characteristics, were incorporated into the models along with site-specific geometric characteristics. The results of this study highlighted the advantages of predicting crash frequencies by crash severities and types separately in winter and summer along with all season. A cross-sectional panel data analysis was carried out for evaluating the safety effectiveness of adding left-turn lanes at four-leg signalized intersections major approaches using RENB models developed in this study. The results indicated a reduction of predicted Fatal+Injury (F+I) and angle crashes by 21.6% and 30.8%, respectively, assuming other factors remain constant. Calibrated models for the winter indicated a reduction of 19.4% and 38.8% for the aforementioned crashes, respectively. The summer crash reduction for these crashes were predicted to be 71% and 69% respectively. The use of the methodological approach which deals with more detailed spatially and temporally correlated crash data, permits a robust understanding of the factors that affect crash frequencies.
INTRODUCTION

In 2015, urban intersections in Wyoming experienced 4024 crashes of which constituted 50% of total urban crashes (Wyoming Department of Transportation, 2015a). According to a study by the National Highway Traffic Safety Administration (NHTSA), left-turning movement was identified as the highest crash-causing pre-crash event, which contributed to 22 percent of all road crashes (Choi, 2010). Left-turn related crashes were estimated as 27 percent of all intersection related crashes in the US and among these, more than two-thirds occurred at signalized intersections (Datta, 2010). Therefore, quantifying the safety efficacy of left-turn lanes at signalized intersections for different crash types and severity levels is essential for better management strategies (Sayed & Leur, 2005). Generally, left-turn related conflicts lead to angle, same direction sideswipe and rear-end crashes (Rodegerdts et al., 2004). Analyzing factors affecting the frequency and severity of the aforementioned crashes can help in better estimation of the effectiveness of specific intersection improvements.

Many researchers have investigated the installation of left-turn lanes at signalized intersection for years in various states in the U.S. A study performed in Nebraska developed Safety Performance Functions for signalized rural and suburban intersections using data from 1988 through 2000 (Khattak, 2010). Negative Binomial regression models were used to accommodate the overdispersion of crash data. The results suggested that natural log transformed traffic volume, area type, roadway horizontal and vertical alignment, and type of left-turn lane (offset left-turn lane) have statistically significant effects on the frequency of intersection approach crashes. The model showed increased crash frequency with the increase in natural logarithm of the exposure (i.e., Total Entering Vehicles). While the presence of offset left-turn lanes decreased the number of crashes, raised medians on minor approaches contributed to an increase in the number of expected crashes on the minor approach. The study also investigated crash rates by crash types. The results of this study indicated that rear-end crashes have the highest rate among all crash types on major approaches at four-leg signalized intersections. A study by Lyon et al. was conducted in Canada to develop SPFs for urban signalized intersections (Lyon, Haq, Persaud, & Kodama, 2005). Crash data obtained from 1950 urban signalized intersections, from 1996 to 2000, were used to develop SPFs for different severity levels. The analyses showed that adding a left-turn lane at four-leg signalized intersections major approaches was significant to reduce PDO crashes. In addition, the effect of adding left-turn lanes on F+I crashes was found to have a low insignificance on 85 percent significance level. A study by Harwood et al. found that installation of a left-turn lane on one major-road approach at signalized intersections reduces total crashes by 18 percent. A reduction of 33 percent of total crashes was found when left-turn lanes are installed on both major-road approaches (Harwood et al., 2002).

The safety effects of roadway treatments and geometric changes in planning, design, operation, and maintenance can be quantified through developing Crash Modification Factors (CMFs). The Cross-sectional method has been widely adopted by many researchers for estimating CMFs. A Cross-sectional study is an observational study which use data at a specific point in time and compares the safety performance of a group of sites with the treatment of interest to similar sites without the treatment (Wu, Lord, & Zou, 2015). Generally, a short period of time (one or two years) is considered for cross-sectional aggregated data because of neglecting time effects. An efficient analysis requires data for a practical period of time to reach a desired level of accuracy. The FHWA recommended that multiple year data should be used to develop SPFs (Herbel, Laing, & McGovern, 2010). Tarko et al. (1998) stated that time trends should be considered for a longer period of analysis; neglecting time trends in longer period of time may results in biased regression parameter estimates (Tarko, Eranky, & Sinha, 1998). According to Lord et al., with many years of data, it is necessary to account for the year-to-year variation, or trend in accident counts because of the influence of factors that change every year (Lord & Persaud, 2000). Assembling multiple years of data over the same set of sites forms panel data which is also known as time-series cross-sectional data or repeated measures (Kweon & Lim, 2012). According to Kweon and Lim, the overdispersion parameter is underestimated in SPFs developed using aggregated data rather than considering panel data (Kweon & Lim, 2012). Again, panel data facilitates multiple observations on each unit which can provide superior estimates as compared to traditional cross-sectional models of association (M. Abdel-Aty & Wang, 2006).

Many studies have provided methodological concepts to predict intersection crashes. Some of the most commonly applied statistical models to accident data include: binomial, Poisson, Negative Binomial, Zero-Inflated Poisson (ZIP), Zero-Inflated Negative Binomial (ZINB), and multinomial probability models. A study using highway safety database administered by the FHWA, demonstrated that the conventional linear regression models lack the
distributional property to describe adequately the random, discrete, nonnegative and typically sporadic accident events on the road (Miaou & Lord, 2003). The use of Poisson regression model has some desirable statistical properties in developing relationships but it may overstate or understate the likelihood of crashes for overdispersed data (El-Basyouny, 2011). Data with overdispersion fit negative binomial (NB) distribution which is a common method in modelling crash frequencies at signalized intersections. Several researchers such as Miaou, Poch & Mannering and Abdel-Aty have used negative binomial regression to overcome the deficiencies of the Poisson regression approach (M. A. Abdel-Aty & Radwan, 2000; Miaou, 1994; Poch & Mannering, 1996). These studies have overcome the overdispersion issue using discrete and non-negative data. Effective safety evaluation strategy is to develop predictive models that are capable of accounting for the different variations and properties which exist in crash data. Multiple years of data for predicting crash frequency of a specific site introduces temporal correlation as many unobserved effects will remain the same over the period. Panel data structure may have a spatial or temporal correlation or may have both and these correlation can be addressed through random effect and fixed effect models (Lord & Mannering, 2010). In an attempt to account for the temporal correlation, some studies have focused on developing crash models using panel data (Chin & Quddus, 2003; Greibe, 2003; Lord & Persaud, 2000). Hausman et al. was the first to apply fixed effect and random effect models for panel count data (Hausman, Hall, & Griliches, 1984) which has been widely adopted in modelling crash data. Random effect and fixed effect models account for unobserved heterogeneity in the crash data (Yaacob, Lazim, & Wah, 2011). Shankar et al. has first employed this application in traffic accident studies (Shankar, Albin, Milton, & Mannering, 1998). The authors stated that the RENB is more appropriate in this field, because geometric and traffic variables are likely to have location-specific effects on crashes. The study also observed a significant improvement of the explanatory power of accident models. A study by Chin & Quddus used RENB model to account for heterogeneity in accident data and investigate the relationship between accident occurrence and the geometric, traffic and control characteristics of signalized intersections in Singapore (Chin & Quddus, 2003). The results showed significant impact of 11 variables on intersection safety which included total approach volume, the numbers of phases per cycle and the left-turn lane as highly confounding factors. An advantage of random effect modeling is that time invariant variables can be included whereas these variables are absorbed in fixed effect modeling (Torres-Reyna, 2007). According to Jiang et al., geometric characteristics and traffic variables are likely to have location-specific effect on crash frequencies (Jiang, Huang, Zaretzki, Richards, & Yan, 2013) which are time-invariant. Therefore, RENB models outperformed traditional negative binomial models when spatial and temporal effects are unobserved (Shankar et al., 1998).

2 STUDY OBJECTIVE

The main objective of this study is to estimate the safety effectiveness of left-turn lanes at four-leg signalized intersections incorporating time trends and taking a longer time frame into consideration. The objective of this study was achieved by developing Wyoming-specific Safety Performance Functions (SPFs) for 174 four-leg signalized intersections using panel data from the years 2005 through 2014. Observed crash frequencies were fitted using NB and RENB models and their performances were compared using the Akaike Information Criterion and log likelihood values. Comparison of the models using panel data structure may account for sites unobserved heterogeneity which may have some influence on the crash frequency. The best model was used to evaluate the safety impact of left-turn lanes at four-leg signalized intersections using the cross-sectional method for total, F+I and PDO crashes including left-turn related crashes (e.g., Angle and Rear-end crashes) separately in winter and summer seasons.

3 METHODOLOGY

CROSS SECTIONAL STUDIES

The effectiveness of countermeasures has been evaluated by different types of cross-sectional studies over the years. Cross-sectional studies for developing CMFs are categorized into regression, case control and cohort methods. Regression method is the simplest and hence were used more frequently in recent studies (Wu et al., 2015). The HSM also suggested using regression models to compare crash frequencies or rates between sites with and without a safety countermeasure (AASHTO, 2010). Developed regression models are used to quantify the impact of the variable representing countermeasure on crash frequencies and CMFs are derived from the respective coefficients (Gross, Persuad, & Lyon, 2010). A cross-sectional study does not need the time for intervention of the treatment (Sasidharan, 2011). Cross-sectional studies’ two main steps are; 1) develop a predictive model, and 2) quantify the safety impacts of highway improvements (Tarko et al., 1998). Approaches used to develop predictive models in the
past are; conventional linear regression model, Poisson regression model, negative binomial regression model, and zero inflated count model.

NEGATIVE BINOMIAL REGRESSION MODEL WITH FIXED AND RANDOM EFFECTS

A traditional cross-sectional model ignores panel data structure as it takes the average crash frequency over the study period per year. Due to this process, there is no account of evidence of variability due to time (M. Abdel-Aty & Wang, 2006). Panel data can control for variables which cannot be observed or measured for the characteristics that may change over time and across entities (M. Abdel-Aty & Wang, 2006). Data used in this study have a panel data structure which have some time-variant and some time-invariant predictors. Time-invariant predictors include geometric characteristics of the intersections. There are also some other unobserved factors for the sites as the intersections were chosen from 23 cities of Wyoming and each city has its own characteristics including population, geography, land use patterns which were not included due the unavailability of data. Obviously, these site-specific heterogeneity have influence over the crash frequency. It is worth mentioning that the intersections have more heterogeneous traffic than a roadway segment. Hence, crash frequency varies largely due to these unobserved variabilities. The fixed effect and random effects approaches are two different ways to introduced unobserved heterogeneity with panel data (Mannering, Shankar, & Bhat, 2016). Fixed effect model assumes the common unobserved effects are accounted for by explanatory variables. Also, the shared unobserved effects of the intersections are assumed to be correlated with independent variable in fixed effect approach. Random effect model assumes that the common unobserved effects are distributed over temporal units according to some distribution. It also assumes that the shared unobserved effects of the intersections are independent of the explanatory variables (Lord & Mannering, 2010).

Traditional Negative Binomial (NB), Fixed Effect Negative Binomial (FENB), and Random Effect Negative Binomial (RENB) model approaches were fitted for total crashes, injury crashes, angle crashes, and rear-end crashes incorporating the variables used for the pooled data. According to Stock and Watson, the key insight of fixed effect is that the unobserved variables do not change over time (Stock & Watson, 2007). Hence, the effect of time-invariant variable is unidentifiable in FENB models. One of the goals of this study was to identify cross-panel association between the crash frequencies and left-turn lanes (time-invariant) in order to evaluate the safety effectiveness of a treatment. Therefore, this goal can be better attained using RENB models rather than the FENB models.

The negative binomial distribution is used to model count data. However, unlike the Poisson distribution, where it is assumed that the mean is equal to the variance, the negative binomial distribution compensates for situations where the variance is greater than the mean, or when the data is overdispersed. Overdispersion for unobserved or unmeasured heterogeneity is addressed. The negative binomial (Poisson-Gamma) model is obtained by the assumption given by equation (1).

\[ e^{(μ_i)}k \sim \text{Gamma}(k, k) \]  

(1)

where, \( k \) is the inverse dispersion parameter. The dispersion (or over-dispersion) parameter is usually referred to as \( β = 1/k \). The probability density function of the NB model is given by equation (2) (El-Basyouny, 2011).

\[ Pr(Y_i = y_i | μ_i, k) = \frac{Γ(y_i+k)}{y_i! Γ(k)} \left( \frac{k}{k+μ_i} \right)^k \left( \frac{μ_i}{k+μ_i} \right)^{y_i} \]  

(2)

Under the NB model, the mean and variance are given by equation (3).

\[ E(Y_i) = μ_i, \quad Var(Y_i) = μ_i + \frac{μ_i^2}{κ} \]  

(3)

When mean will be equal to variance, \( β \) will go to zero and NB model would correspond to the Poisson model. The Negative Binomial regression model has been widely applied in the road safety literature.

The RENB model addresses random effects by introducing a term, \( v_i \) into the relationship between the expected number of crashes (\( μ_{i,t} \)) and the covariates, \( X_{i,t} \), of an intersection site \( i \) in a given time period \( t \) (SAS Institute Inc., 2013),

\[ μ_{i,t} = μ_i v_i = e^{(X_{i,t}β + η_{i,t})} \]  

(4)
where, \( v_i \) is the random effects; \( \beta \) is a vector of estimable parameters, \( e^{\eta t} \) is a gamma distributed error term with mean 1 & variance \( \kappa \). In order to account for the variation of location over time, the distribution of a function of \( v_i \) is assumed as beta with parameters \((a, b)\),

\[
\frac{v_i}{1 + v_i} \sim Beta (a, b) \tag{5}
\]

The RENB conditional distribution function for the \( i^{th} \) intersection is given by equation (6).

\[
P[y_{i1}, ..., y_{iT}|x_{i1}, ..., x_{iT}] = \frac{\Gamma(a + b) \Gamma(a + \sum_{t=1}^{T_i} \mu_{it}) \Gamma(b + \sum_{t=1}^{T_i} y_{it})}{\Gamma(a) \Gamma(b) \Gamma(a + b + \sum_{t=1}^{T_i} \mu_{it} + \sum_{t=1}^{T_i} y_{it})} \times \prod_{t=1}^{T_i} \frac{\Gamma(\mu_{it} + y_{it})}{\Gamma(\mu_{it}) \Gamma(y_{it} + 1)} \tag{6}
\]

The model goodness of fit is also examined using the Akaike Information Criterion (AIC) and log likelihood values.

**EVALUATION OF THE SAFETY EFFECTIVENESS OF ADDING LEFT-TURN LANES**

Let, LL is the variable resembling left-turn lanes at intersections. Presence of LL at intersection is denoted as \( LL = 1 \), and 0 denoted as the intersection without LL. SPFs for intersections including LL as a variable can be expressed as,

\[ Y_i = e^{(\beta_0 + \beta_1 \times LL)} \tag{7} \]

For the sites with and without LL, the resulting equation can be shown by assigning \( LL = 1 \) and \( LL = 0 \) in equation (7) respectively.

For \( LL = 1 \),

\[ Y_i = e^{(\beta_0 + \beta_1 \times 1)} \]

For \( LL = 1 \),

\[ Y_i = e^{(\beta_0)} \]

The crash modification factor for adding left-turn lanes can be inferred by,

\[ CMF = \frac{e^{(\beta_0 + \beta_1)}}{e^{(\beta_0)}} \]

**4 DATA DESCRIPTION**

Total 174 four-leg signalized intersections from 23 cities within 20 counties in the State of Wyoming were chosen as the study sites considering the availability of traffic volume data. All raw crash data of Wyoming were collected for the years 2005-2014 from the Critical Analysis Reporting Environment (CARE), a crash database developed by the Wyoming Department of Transportation (WYDOT). These data were imported into Geographic Information System mapping tool to assign intersection related crashes to intersections. The intersection influence area should be defined to classify the intersection related crashes. The intersection safety influence area depends on the intersection geometry, traffic control, and operating features (Tarko et al., 1998). The State of Indiana among other states used a circular influence area of 250 ft. radius from the center of the intersection (Wu et al., 2015). Channelized intersections influence area was defined within 20 ft. beyond the gore of islands or the point at which the turn lane attains (Milton & Mannering, 1998). The safety effects could be overestimated if a larger safety influence area is applied to smaller intersections and hence misclassifying roadway segment crashes as intersection crashes (Tarko et al., 1998). In this study, 250 ft. criterion was used to define intersection influence area. A number of 12,365 crashes
were found to be intersection related crashes from 2005 through 2014. Crash data by severity (i.e., total, F+I, and PDO), and collision types (i.e., angle, rear-end) were categorized from the obtained data.

Intersections consisting of at least one collector road were considered for selecting sites, so that traffic volume data for at least major approach could be collected from WYDOT. Typically, traffic volume estimates for local road sections without traffic counts and AADT are made by comparing a road section to other similar road sections without traffic count data (Mohamad, Sinha, Kuczek, & Scholer, 1998). In case of unavailability of minor approach traffic volume data in this study, AADTs were assumed as a percentage of major roadway AADT using roadway geometry and land use information. Summer and winter AADTs were obtained by calculating seasonal factors using Monthly Average Daily Traffic (MADT) of the corresponding months of winter and summer. As observed MADT variations were quite similar over the years from 2004 to 2014; 2009 was taken as the representative year for calculating seasonal factors for 20 counties of Wyoming. The average seasonal factors for traffic volume were calculated as 0.42 and 0.58 for winter and summer respectively. Intersection geometric characteristics were not available in Wyoming roadway inventory. Geometric characteristics data were extracted by inspecting satellite imageries in Google Earth Pro®. For each intersection, number of through lanes, intersection skew angles, presence of raised median, presence of on-street parking and number of left turn and right turn lanes were obtained accurately. Therefore, an extensive manual data collection was a challenging task for this study.

Weather is considered a prime factor contributing to crash frequencies in Wyoming. Weather data were collected from the National Oceanic and Atmospheric Administrations (NOAA) weather stations. Weather stations maintained by airports were found to be the most reliable having at least 80% complete data of the study period and were considered in this study. Number of rainy and snowy days for each intersection were collected for the intersections within five nautical miles buffer zone around the airport stations (Ahmed et al. 2014).

The Wyoming Strategic Highway Safety Plan (2012) and Wyoming’s Highway Safety Plan (2015) emphasized on impaired and young drivers each of which produces high number of crashes consistently (Wyoming Department of Transportation, 2015b; Wyoming Highway Safety Management System Committee, 2012). Therefore, these drivers’ characteristics in crash prediction models were included to determine the negative impact of drivers behavior on traffic safety. Data for young and impaired drivers involved in crashes in the study intersections were collected from CARE package. As, the number of licensed drivers were different in each year, young and impaired drivers counts were normalized with the number of Wyoming licensed drivers for the corresponding years. Wyoming licensed drivers count by age were obtained from the ‘Highway Statistics Series’ of the FHWA (Federal Highway Administration, 2014).

<table>
<thead>
<tr>
<th>Continuous Variables</th>
<th>Average</th>
<th>Min.</th>
<th>Max.</th>
<th>Std. Dev.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Major AADT: Annual Average Daily Traffic in major approach roadway (veh/day)</td>
<td>13610.93</td>
<td>346.00</td>
<td>38120.00</td>
<td>5936.05</td>
</tr>
<tr>
<td>Minor AADT: Annual Average Daily Traffic in minor approach roadway (veh/day)</td>
<td>6555.84</td>
<td>207.60</td>
<td>31258.40</td>
<td>4800.64</td>
</tr>
<tr>
<td>Log Major AADT: logarithm of Major AADT</td>
<td>9.41</td>
<td>5.85</td>
<td>10.55</td>
<td>0.51</td>
</tr>
<tr>
<td>Log Minor AADT: logarithm of Major AADT</td>
<td>8.51</td>
<td>5.34</td>
<td>10.35</td>
<td>0.80</td>
</tr>
<tr>
<td>Total crashes per year per intersection</td>
<td>7.11</td>
<td>0.00</td>
<td>60.00</td>
<td>6.76</td>
</tr>
<tr>
<td>F+I crashes per year per intersection</td>
<td>1.68</td>
<td>0.00</td>
<td>18.00</td>
<td>2.11</td>
</tr>
<tr>
<td>PDO crashes per year per intersection</td>
<td>5.43</td>
<td>0.00</td>
<td>42.00</td>
<td>5.22</td>
</tr>
<tr>
<td>Angle crashes per year per intersection</td>
<td>2.30</td>
<td>0.00</td>
<td>19.00</td>
<td>2.64</td>
</tr>
<tr>
<td>Rear-end crashes per year per intersection</td>
<td>2.49</td>
<td>0.00</td>
<td>29.00</td>
<td>3.28</td>
</tr>
<tr>
<td>Number of Rainy Days per year per intersection</td>
<td>75.28</td>
<td>5.00</td>
<td>169.00</td>
<td>27.31</td>
</tr>
<tr>
<td>Number of Snowy Days per year per intersection</td>
<td>35.27</td>
<td>2.75</td>
<td>349.00</td>
<td>43.91</td>
</tr>
<tr>
<td>Young Drivers Proportion per year per intersection: young driver’s involvement in crash; Normalized by registered licensed drivers in Wyoming aged below 25 of corresponding year.</td>
<td>3.00</td>
<td>0.00</td>
<td>7.54</td>
<td>2.03</td>
</tr>
<tr>
<td>Impaired drivers’ Proportion per year per intersection: crash proportions for impaired driving</td>
<td>5.34</td>
<td>0.00</td>
<td>100.00</td>
<td>14.72</td>
</tr>
</tbody>
</table>

Note: Traffic volume and crashes vary along time, and this table was calculated based on 1740 observations.
Table 2. Descriptive statistics for categorical variables

<table>
<thead>
<tr>
<th>Categorical Variables</th>
<th>Levels</th>
<th>Number of Intersections</th>
<th>Percent of Intersections</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of through lanes at major approach</td>
<td>0, if less than 5 lanes</td>
<td>154</td>
<td>88.5%</td>
</tr>
<tr>
<td></td>
<td>1, if more than 4 lanes</td>
<td>20</td>
<td>11.5%</td>
</tr>
<tr>
<td>Number of through lanes at minor approach</td>
<td>0, if less than 5 lanes</td>
<td>19</td>
<td>89.0%</td>
</tr>
<tr>
<td></td>
<td>1, if more than 4 lanes</td>
<td>155</td>
<td>11.0%</td>
</tr>
<tr>
<td>Presence of left-turn lanes at major approach</td>
<td>0, if left-turn lane is present</td>
<td>144</td>
<td>82.8%</td>
</tr>
<tr>
<td></td>
<td>1, if left-turn lane is absent</td>
<td>30</td>
<td>17.2%</td>
</tr>
<tr>
<td>Presence of left-turn lanes at minor approach</td>
<td>0, if left-turn lane is present</td>
<td>115</td>
<td>66.1%</td>
</tr>
<tr>
<td></td>
<td>1, if left-turn lane is absent</td>
<td>59</td>
<td>33.9%</td>
</tr>
<tr>
<td>Presence of right-turn lanes at major approach</td>
<td>0, if right-turn lane is present</td>
<td>123</td>
<td>70.7%</td>
</tr>
<tr>
<td></td>
<td>1, if right-turn lane is present</td>
<td>55</td>
<td>29.3%</td>
</tr>
<tr>
<td>Presence of right-turn lanes at minor approach</td>
<td>0, if right-turn lane is absent</td>
<td>119</td>
<td>68.4%</td>
</tr>
<tr>
<td></td>
<td>1, if raised median is present</td>
<td>17</td>
<td>9.8%</td>
</tr>
<tr>
<td>Presence of raised median at major approach</td>
<td>0, if raised median is absent</td>
<td>157</td>
<td>90.2%</td>
</tr>
<tr>
<td></td>
<td>1, if raised median is present</td>
<td>4</td>
<td>2.3%</td>
</tr>
<tr>
<td>Presence of on-street parking in any approach</td>
<td>0, if on-street parking is absent</td>
<td>77</td>
<td>44.3%</td>
</tr>
<tr>
<td></td>
<td>1, if on-street parking is present</td>
<td>97</td>
<td>55.7%</td>
</tr>
<tr>
<td>Skew angle of the intersection</td>
<td>0, if greater than 60 degree angle</td>
<td>169</td>
<td>98.1%</td>
</tr>
<tr>
<td></td>
<td>1, if smaller than 60 degree angle</td>
<td>5</td>
<td>2.9%</td>
</tr>
</tbody>
</table>

Note: All categorical variables are time constant, this table was calculated based on 174 intersections.

The data were arranged including the five aforementioned data sets for the panel structure. Total observation for panel data became (174 intersections over 10 years) 1740. Intersection crash frequencies, traffic volumes, young and impaired driver’s proportion and number of rainy and snowy days varied from year to year and all other variables remained constant during the study period. Descriptive statistics of the data incorporated in the models are shown in Table 1 & Table 2 for continuous and categorical variables respectively.

DATA TRENDS BY WEATHER AND CRASH TYPES

Wyoming has Rocky Mountains and Plain Regions’ unique weather characteristics which results in long winters in comparison to most of the US states (Ruffner, 1985). Crash frequencies by severity and collision types and their proportions by summer and winter from 2005 to 2014 of these sites are reported in Figure 1. Summer data were taken from mid-April through mid-October and winter data were taken from mid-October through mid-April.

![Crash frequency and proportions in winter and summer for observed crashes at study sites](image)

Winter and summer crash factors are significantly different in Wyoming by crash types. Crash frequencies of different crash types also changes with the variations from cold to mild periods. Therefore, developing Safety Performance Functions in all season, winter & summer for angle and rear-end crashes along with injury and total crashes satisfy the focus of this research. Hence a cross sectional analysis was also done to evaluate the safety effectiveness of adding left-turn lanes in general, winter & summer condition for the specific types of crashes mentioned above.
CRASH RELATION TO DRIVERS AGE

Crash involvement rates were observed higher for drivers’ age less than 20 and decrease invariably with increasing age until 70, and increase slightly thereafter. Tefft stated that the involvement of drivers more than 70 years old is the same as drivers in their 30’s in terms of number of crashes per mile driven (Tefft, 2012). A similar result is obtained in this study. Figure 2 illustrates the percentage of licensed drivers of Wyoming by certain age range and compares with the number of crashes per 10,000 licensed drivers of within corresponding age range. It is observed that drivers’ ages below 25 has the highest rate of crashes and follow a different distribution than the other age groups. Crash rate decreased with a flat distribution until drivers ages 60-69. Seventy and older drivers showed a little increase thereafter. Therefore, crash prediction models were developed including the involvement of young drivers (<25) which can be a confounding factor for crash occurrence.

Figure 2. Total crash per 10,000 licensed drivers by age group for observed crashes at study sites

5 RESULTS

MODELING CRASH DATA

Traditional NB and RENB model approaches were fitted for total crashes, injury crashes, angle crashes and rear-end crashes incorporating the variables shown in Table 1 and Table 2. The fitted models are provided in Table 3 comparing NB and RENB for crash severities and collision types for all season. Least AIC value was considered to select the best performed models. AIC values were compared and found that AIC value is least in the RENB distribution for all the models. It represents that panel data structure fit in RENB model better than a traditional NB model. Therefore, separate models for winter and summer were developed using RENB and shown in Table 4 and Table 5 respectively for all crash types. Variable selection for the models was made by p-values. Several studies using negative binomial regression model with cross-sectional and panel data considered variables up to 80 percent significant level (M. Abdel-Aty & Wang, 2006; Bauer & Harwood, 2000; Tegge, Jo, & Ouyang, 2010). Statistically insignificant variables (p-values > 0.20) were removed manually one at a time until only significant variables remained in the model. Hence, all the reported variables were significant at 80% or higher significant level.

The variable estimates for all crash severity and collision types showed trends for that specific type or severity. The variable estimates in the RENB models are consistent with those in NB models. However, NB models overestimated the impacts slightly higher than RENB models. Presence of left-turn lanes in major approaches has positive effect on crash reduction in all models. More than 3 lanes in intersection legs showed increased crashes than having 3 or 2 lanes both for major and minor approaches. More lanes lead to more traffic hence increase the probability of collision. Similarly, increase in Major and Minor Approach AADT indicated more crashes. Wyoming’s legislature prohibit on-street parking within 20 feet (6.1 meter) of a crosswalk at an intersection (Wyoming Statutes, 2016). A study in New Zealand mentioned that signalized intersections with parking within 30-40 meter of the limit line are associated with a high number of crashes (Turner, Singh, & Nates, 2012). But another study stated that higher approach speed at four-leg signalized intersections in Singapore increased intersection related crashes (Mitra, Chin, & Qudus, 1999). In this study, on-street parking showed decrease in all types of crashes in all the models. But F+I and angle crash reduction due to on-street parking showed 10 percent more than other crash types. Crash reduction in effect of on-street parking is unexpected whereas it can be assumed that lower approach speed due to presence of on-street parking prevents crashes at intersection.
Table 3. Parameter estimates for NB and RENB models for all season crashes

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Total Crash Estimates</th>
<th>F+I Crash Estimates</th>
<th>PDO Crash Estimates</th>
<th>Angle Crash Estimates</th>
<th>Rear-end Crash Estimates</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>NB</td>
<td>RENB</td>
<td>NB</td>
<td>RENB</td>
<td>NB</td>
</tr>
<tr>
<td>Major AADT</td>
<td>0.643</td>
<td>0.536</td>
<td>0.648</td>
<td>0.627</td>
<td>0.64</td>
</tr>
<tr>
<td>Minor AADT</td>
<td>0.2</td>
<td>0.174</td>
<td>0.211</td>
<td>0.263</td>
<td>0.197</td>
</tr>
<tr>
<td>Major Lanes</td>
<td>N/A</td>
<td>N/A</td>
<td>0.187*</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Minor Lanes</td>
<td>0.326</td>
<td>0.343</td>
<td>0.441</td>
<td>0.406</td>
<td>0.288</td>
</tr>
<tr>
<td>Major Left-turn</td>
<td>-0.358</td>
<td>-0.283</td>
<td>-0.248</td>
<td>-0.243</td>
<td>-0.398</td>
</tr>
<tr>
<td>Minor Left-turn</td>
<td>0.201</td>
<td>0.232</td>
<td>0.261</td>
<td>0.313</td>
<td>0.172</td>
</tr>
<tr>
<td>Major Right-turn</td>
<td>0.222</td>
<td>N/A</td>
<td>0.199</td>
<td>0.173*</td>
<td>0.245</td>
</tr>
<tr>
<td>Minor Right-turn</td>
<td>0.313</td>
<td>0.41</td>
<td>0.309</td>
<td>0.343</td>
<td>0.311</td>
</tr>
<tr>
<td>Major Median</td>
<td>0.147</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>0.169</td>
</tr>
<tr>
<td>Minor Median</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>On-Street Parking</td>
<td>-0.271</td>
<td>-0.241</td>
<td>-0.4</td>
<td>-0.407</td>
<td>-0.229</td>
</tr>
<tr>
<td>Skew Angle</td>
<td>-0.233</td>
<td>N/A</td>
<td>-0.291*</td>
<td>N/A</td>
<td>-0.192*</td>
</tr>
<tr>
<td>Rain</td>
<td>0.002</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Snow</td>
<td>-0.001*</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Young Driver</td>
<td>0.049</td>
<td>0.046</td>
<td>0.077</td>
<td>0.079</td>
<td>0.034*</td>
</tr>
<tr>
<td>Impaired Driver</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>AIC</td>
<td>9347.04</td>
<td>7814</td>
<td>5445.26</td>
<td>5290</td>
<td>8625.03</td>
</tr>
<tr>
<td>Log Likelihood</td>
<td>9317.04</td>
<td>8692</td>
<td>5419.26</td>
<td>5266</td>
<td>8593.03</td>
</tr>
</tbody>
</table>

Note: N/A- not available; *p<0.10; **p<0.15; ***p<0.20

All the models showed negative impact of increasing young driver (ages <25) involvement on crashes. Impact of young drivers’ involvement on crash frequencies is projected in the models which is in line with the descriptive analysis of this study. This result implies that young drivers are more prone to high crash risk relative to elderly and more experienced drivers. Impaired driving or driving under the influence increased crash frequency in winter crash models but was insignificant for all season models. It is anticipated from this result that driving under the influence in winter condition is more crash prone. But impact of young drivers’ involvement showed increase in F+I and angle crashes in winter than in summer. It can be anticipated that young drivers are more vulnerable to winter crashes which is also observed from crash statistics (Minnesotans for Safe Driving, 2015.). Moreover, Mueller et al. found that the young novice drivers have the longest hazard response time and the smallest speed compensation for fog (Mueller & Trick, 2012). Number of rainy days’ estimates showed increased predicted PDO and angle crash frequencies in all season models but in winter models it showed insignificance except for angle crashes. It is obvious because the rainfall trends in Wyoming show that the number of average days with precipitation during the winter months is 6 days with an average precipitation of less than 1 inch (U.S. Climate Data, 2017). Therefore, rainfall doesn’t have any impact on crashes in winter.

Table 4. Parameter estimates for winter crashes

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Total Crash Estimates</th>
<th>F+I Crash Estimates</th>
<th>PDO Crash Estimates</th>
<th>Angle Crash Estimates</th>
<th>Rear-end Crash Estimates</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>NB</td>
<td>P-Value</td>
<td>NB</td>
<td>P-Value</td>
<td>NB</td>
</tr>
<tr>
<td>Intercept</td>
<td>-4.879</td>
<td>&lt;.0001</td>
<td>-6.714</td>
<td>&lt;.0001</td>
<td>-4.693</td>
</tr>
<tr>
<td>Major AADT</td>
<td>0.675</td>
<td>&lt;.0001</td>
<td>0.783</td>
<td>&lt;.0001</td>
<td>0.664</td>
</tr>
<tr>
<td>Minor AADT</td>
<td>0.133</td>
<td>0.018</td>
<td>0.171</td>
<td>0.020</td>
<td>0.134</td>
</tr>
<tr>
<td>Major Lanes</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Minor Lanes</td>
<td>0.266</td>
<td>0.021</td>
<td>0.313</td>
<td>0.023</td>
<td>0.294</td>
</tr>
<tr>
<td>Major Left-turn</td>
<td>-0.357</td>
<td>0.001</td>
<td>-0.216</td>
<td>0.119</td>
<td>-0.405</td>
</tr>
<tr>
<td>Minor Left-turn</td>
<td>0.208</td>
<td>0.016</td>
<td>0.217</td>
<td>0.057</td>
<td>0.194</td>
</tr>
<tr>
<td>Major Right-turn</td>
<td>N/A</td>
<td>N/A</td>
<td>0.190</td>
<td>0.076</td>
<td>0.130</td>
</tr>
<tr>
<td>Minor Right-turn</td>
<td>0.391</td>
<td>&lt;.0001</td>
<td>0.384</td>
<td>0.000</td>
<td>0.353</td>
</tr>
<tr>
<td>Major Median</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>On-Street Parking</td>
<td>-0.258</td>
<td>0.001</td>
<td>-0.408</td>
<td>&lt;.0001</td>
<td>-0.202</td>
</tr>
<tr>
<td>Rain</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Young Driver</td>
<td>0.045</td>
<td>&lt;.0001</td>
<td>0.028</td>
<td>0.080</td>
<td>0.048</td>
</tr>
<tr>
<td>Impaired Driver</td>
<td>0.003</td>
<td>0.026</td>
<td>0.005</td>
<td>0.009</td>
<td>N/A</td>
</tr>
<tr>
<td>AIC</td>
<td>7199</td>
<td>3719</td>
<td>6606</td>
<td>4838</td>
<td>6467</td>
</tr>
<tr>
<td>SBC</td>
<td>7264</td>
<td>3796</td>
<td>6672</td>
<td>4909</td>
<td>4740</td>
</tr>
<tr>
<td>Log Likelihood</td>
<td>-3587</td>
<td>-1846</td>
<td>-3291</td>
<td>-2406</td>
<td>-2325</td>
</tr>
</tbody>
</table>
Table 5. Parameter estimates for summer crashes

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Total Crash Estimates</th>
<th>Fatal+Injury Crash Estimates</th>
<th>PDO Crash Estimates</th>
<th>Angle Crash Estimates</th>
<th>Rear-end Crash Estimates</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>P-Value</td>
<td>P-Value</td>
<td>P-Value</td>
<td>P-Value</td>
<td>P-Value</td>
</tr>
<tr>
<td>Intercept</td>
<td>-4.907</td>
<td>-6.693</td>
<td>-4.468</td>
<td>-5.251</td>
<td>-6.061</td>
</tr>
<tr>
<td>Major AADT</td>
<td>&lt;.0001</td>
<td>&lt;.0001</td>
<td>&lt;.0001</td>
<td>&lt;.0001</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>Minor AADT</td>
<td>0.523</td>
<td>0.614</td>
<td>0.481</td>
<td>0.501</td>
<td>0.624</td>
</tr>
<tr>
<td>Major Lanes</td>
<td>0.227</td>
<td>0.252</td>
<td>0.247</td>
<td>0.284</td>
<td>0.357</td>
</tr>
<tr>
<td>Minor Lanes</td>
<td>0.232</td>
<td>0.226</td>
<td>0.166</td>
<td>0.217</td>
<td>0.243</td>
</tr>
<tr>
<td>Major Left-turn</td>
<td>0.307</td>
<td>0.429</td>
<td>0.309</td>
<td>0.383</td>
<td>0.343</td>
</tr>
<tr>
<td>Minor Left-turn</td>
<td>-0.364</td>
<td>-0.343</td>
<td>-0.395</td>
<td>-0.371</td>
<td>-0.263</td>
</tr>
<tr>
<td>Major Right-turn</td>
<td>0.283</td>
<td>0.194</td>
<td>0.304</td>
<td>0.249</td>
<td>0.324</td>
</tr>
<tr>
<td>Minor Right-turn</td>
<td>0.274</td>
<td>0.251</td>
<td>0.243</td>
<td>0.286</td>
<td>0.306</td>
</tr>
<tr>
<td>Major Median</td>
<td>0.213</td>
<td>0.191</td>
<td>0.195</td>
<td>0.175</td>
<td>0.175</td>
</tr>
<tr>
<td>On-Street Parking</td>
<td>-0.380</td>
<td>-0.380</td>
<td>-0.167</td>
<td>-0.276</td>
<td>-0.239</td>
</tr>
<tr>
<td>Rain</td>
<td>0.002</td>
<td>0.019</td>
<td>-0.002</td>
<td>-0.017</td>
<td>-0.020</td>
</tr>
<tr>
<td>Snow</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Young Driver</td>
<td>-0.007</td>
<td>0.011</td>
<td>-0.013</td>
<td>0.010</td>
<td>0.016</td>
</tr>
<tr>
<td>Impaired Driver</td>
<td>0.005</td>
<td>0.006</td>
<td>0.002</td>
<td>0.004</td>
<td>0.004</td>
</tr>
<tr>
<td>AIC</td>
<td>7085</td>
<td>4111</td>
<td>6405</td>
<td>4564</td>
<td>4736</td>
</tr>
<tr>
<td>SBC</td>
<td>7173</td>
<td>4184</td>
<td>6498</td>
<td>4646</td>
<td>4813</td>
</tr>
<tr>
<td>Log Likelihood</td>
<td>-3527</td>
<td>-2042</td>
<td>-3185</td>
<td>-2267</td>
<td>-2354</td>
</tr>
</tbody>
</table>

SAFETY EFFECTIVENESS EVALUATION FOR LEFT-TURN LANES

This paper discussed three distributions for crash prediction models for panel data. The RENB model had better specifications than the traditional NB distributions. A cross-sectional analysis was done to measure the safety effectiveness of adding left-turn lanes, using the Wyoming-specific SPFs developed in this study. From the preliminary analysis, target crashes during summer and winter are shown in Table 6. Winter PDO and Angle crash frequencies are higher than summer crash frequencies by 1.05 and 1.13 times respectively. However, higher crash proportions were expected in winter for F+I, rear-end and total crashes also but these crashes showed slightly higher crash frequency in summer than in winter. In average, 38% higher traffic is observed in summer than in winter in Wyoming. Therefore, summer crashes are slightly overrepresented. Crash rate by TEV during summer and winter were also presented in Table 6. Winter crash rates showed more than twice of the summer crash rates.

Table 6. Percentages of target crashes during summer and winter

<table>
<thead>
<tr>
<th>Crash Frequency</th>
<th>Crash/TEV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Winter</td>
<td>Summer</td>
</tr>
<tr>
<td>Total Crashes</td>
<td></td>
</tr>
<tr>
<td>PDO Crashes</td>
<td></td>
</tr>
<tr>
<td>Angle Crashes</td>
<td></td>
</tr>
<tr>
<td>Rear End Crashes</td>
<td></td>
</tr>
</tbody>
</table>

Different model approaches which were used in this study generated crash modification factors for adding left turn lanes at major approaches showed in Table 7.

Table 7. Safety effectiveness of major approach left-turn lanes at four-leg signalized intersections

<table>
<thead>
<tr>
<th>Parameter</th>
<th>All Season</th>
<th>Winter</th>
<th>Summer</th>
</tr>
</thead>
<tbody>
<tr>
<td>CMF (Safety effectiveness)</td>
<td>Significance</td>
<td>CMF (Safety effectiveness)</td>
<td>Significance</td>
</tr>
<tr>
<td>Total</td>
<td>0.75 (25%)</td>
<td>99.4%</td>
<td>0.70 (30%)</td>
</tr>
<tr>
<td>Fatal+Injury</td>
<td>0.78 (22%)</td>
<td>95.8%</td>
<td>0.81 (19%)</td>
</tr>
<tr>
<td>PDO</td>
<td>0.71 (29%)</td>
<td>99.8%</td>
<td>0.67 (33%)</td>
</tr>
<tr>
<td>Angle</td>
<td>0.69 (31%)</td>
<td>99.8%</td>
<td>0.61 (39%)</td>
</tr>
<tr>
<td>Rear-end</td>
<td>0.73 (27%)</td>
<td>98.1%</td>
<td>0.75 (25%)</td>
</tr>
</tbody>
</table>

Crash modification factors for presence of left-turn lane at major approach of the intersection during winter showed 25% reduction in all season, 30% reduction in winter and 31% reduction in summer for total crashes. Angle crash reduction showed the highest crash reduction (39%) during the winter due to presence of the left-turn lanes.
CMF for this countermeasure showed the highest crash reduction for F+I crashes in summer, which is 29%. Rear-end crashes did not show variation of crash reduction during winter compared to all season.

6 CONCLUSIONS

The purpose of this research is to estimate the safety effectiveness of adding left-turn lanes at four leg-signalized intersections considering time trend over a longer period of time. A 10 years (2005-2014) of panel data in Wyoming were incorporated in NB and RENB models to determine the impacts of geometric, weather and driver characteristics on crash frequency by crash severity and collision types. To explicitly understand the effects of the confounding factors on winter crashes, there was a necessity to perform a safety performance analysis separately for winter and summer seasons. The variable estimates obtained from the separate winter models justified its importance. Left-turn lanes in major approach resulted in a huge amount of predicted angle crash reduction in winter season (39%) compared to all season & summer (31%) model. Winter model for F+I crashes was also found to contain impaired drivers as significant variables to increase crashes while it was not significant in summer model for F+I crashes. Impact of young drivers’ involvement showed an increase in F+I and angle crashes in winter than in summer and all season. Impaired driving was represented as a significant variables in summer models to increase crashes by all types and severity. But it was significant for total, F+I and rear-end crashes in winter.

In this study, traditional Negative Binomial (NB), and NB with Fixed and Random Effects were fitted. The FENB failed to properly explain the association between crash frequency and most of the explanatory variables. The interpretation of FENB models was that an increase in exposure measure (AADT) reduces crashes while the presence of left-turn lanes increases crash frequency at four-leg signalized intersections which is counterintuitive. While the RENB can handle both time-variant and time-invariant variables, the inclusion of a large number of time-invariant variables in the models could be the reason behind the unintuitive results from the FENB. Moreover, the RENB models were found to outperform the traditional NB models using panel data. AIC and log likelihood values were considered as model selection criteria to establish the suitability of RENB models. RENB model accommodated time trend while NB models did not account for any time trend. Random effects models assumed to have the influence of sites heterogeneity on crash frequencies and this influence is not correlated with the independent variables. However, fixed effect approach considers the unobserved heterogeneity of the sites when heterogeneity is constant over time and correlated with independent variables. As RENB model appeared to be the best model in this study, it can be concluded that random effect models are better suited for panel data when site characteristics have random features and unobserved heterogeneity among the sites have influence over the observed crash frequencies (Chin & Quddus, 2003; Kweon & Kockelman, 2004). Future research in this area includes estimation of CMFs for more explanatory variables in multilevel or hierarchical models applying Bayesian before-after approach for more reliable results.

REFERENCES


Torres-Reyna, O. (2007). Panel data analysis fixed and random effects using Stata (v. 4.2). Data & Statistical Services, Princeton University.


KEYWORDS:
traffic signs, road markings, retroreflection, visibility, maintenance

ABSTRACT:
Traffic signs and road markings as part of traffic control systems, inform traffic participants about road conditions, dangers, limitations and other information needed for their safety on the road. As a basic means of communication between the road authorities and traffic participants their improper installation and maintenance may affect the driver’s perception process and the quality of transmitted messages, thus affecting the drivers’ reaction and general traffic safety. Therefore, to efficiently manage a maintenance system of traffic signs and road markings an systematic quality testing is needed. The aim of this paper is to analyse how can this type of approach, based on the results from Croatia, maintain the quality of road markings and traffic signs at the satisfactory level and increase their quality in their service life. Also, this paper presents results of quality testing of traffic signs and road markings last 10 years in the Republic of Croatia with analysis and conclusions which prove their importance in road safety.
Testing the Quality of Traffic Signs and Road Markings: Republic of Croatia Case Study

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1 INTRODUCTION

Road safety presents a complex social problem which causes considerable economic losses to individuals, their families and society. These losses comprise of a direct costs of material damage and medical care as well of indirect costs of police, courts, insurance companies and the loss of productivity for those killed or disabled by their injuries and their family members who need to take time off work or school to care for the injured. According to the calculations made by World Health Organization (WHO), road accidents cost most of the countries up to 3% of their gross domestic product.

Road accidents are the result of the interaction of three factors: humans, vehicle and road and its surrounding. Until recently, it was thought that the human factor is the cause of majority road accidents. Nowadays, majority of scientific and practical community involved in this subject, shift their focus more to road infrastructure and road surrounding as possible cause of traffic accidents. According to a report by the United Nations Road Safety Collaboration “Working Group 4 – Infrastructure”, targeted investment in road infrastructure can generate crash cost savings of up to 60 times the cost of construction which means that for each € 1 invested, there was a return of up to € 60 in terms of crash costs avoided (Safe Roads for Development). Similar research, based on the in-depth analysis of 230 fatal crashes, found that non-compliance with road safety criteria was exclusively responsible for 32.6% of the fatal accident and a contributing factor in 30.8% (Stigson et al. 2008). Based on all above, one can conclude that with improvements in road infrastructure may significantly increase traffic safety.

As a basic means of communication between the road authorities and traffic participants traffic signs and road markings present essential elements of road infrastructure and are one of the most cost-effective safety solutions available to road authorities. But to assure that traffic signs and road markings will fulfil their tasks they need to be properly installed and maintenance. Their improper installation and maintenance may affect the driver’s perception process and the quality of transmitted messages, thus affecting the drivers’ reaction and general traffic safety.

Maintenance of the traffic signs and road markings include several tests focused on the checking their quality. From the traffic safety point of view, the most important tests are tests in which visibility i.e. retroreflection of traffic signs and road markings is measured. The aim of this paper is to present methodology for measuring retroreflection of traffic signs and road markings based on the extensive experience from Republic of Croatia.

2 METHODS FOR MEASURING THE QUALITY OF ROAD MARKINGS

Retroreflection measurements of road markings give the road authorities an inside look at how road markings perform, in which condition they are and how well conductors do their job. With this inside look, road authorities can create a plan of renewal based on measurement results. Roads that, according to measurements results, have satisfying retroreflection will not be renewed when their retroreflection decreases below minimum prescribed levels which may have significant impact on the reduction and planning of the maintenance costs.

In the Republic of Croatia, according to the technical requirements of Croatian Roads ltd. (Guidelines and technical requirements for performing works on renewing road markings, 2010), the conducted tests ensure prescribed quality of road markings which comprise (Babić et al. 2016):

− convenience tests,
− ongoing tests,
− control tests,
− additional control tests,
− arbitrary tests,
− tests prior to the expiry of the warranty.
Convenience tests include tests aimed at proving the convenience or suitability of a material intended to be used for applying road markings, based on the foretold type of marking and prescribed quality. Ongoing tests, conducted by the Works Contractor, determine the prescribed quality of material and works performance. The tests comprise testing of the thickness of wet and dry paint layer, testing daytime and night-time visibility in dry conditions, testing night-time visibility in wet conditions (only for type II road markings – road markings with special properties intended to enhance the retroreflection in wet or rainy conditions) and slip resistance.

Control tests, ensured by the Road Authority, determine whether the quality of the road markings system is compliant with the prescribed requirements. Mentioned tests comprise:

- control tests prior to the application of road markings which include identification, that is, verification of compliance (chemical and physical tests) between the delivered samples of the road markings material and the information presented in the certificates;
- control tests during the application of road markings, which include testing of the drying time, the thickness of wet and dry layer, the quantity of retroreflective material (glass beads) in the material and visual inspection of road markings;
- control tests applied on road markings, which include testing daytime and night-time visibility in dry conditions, testing night-time visibility in wet conditions (only for type II road markings) and slip resistance, as well as testing of the road markings geometry in terms of designed road markings width and length.

Additional control tests are conducted only if control tests on applied road markings resulted in limit values.

Arbitrary tests involve repeating the control tests, if the Road Authority or the Contractor did not conduct the tests appropriately. An authorized legal entity which has not taken part in the disputed tests or which has been approved by both parties conducts the tests in order to determine the quality of applied road markings and its compliance with the quality agreed for the duration of the warranty period.

One of the most important elements when evaluating the quality of road markings is testing their daytime and night-time visibility. The results of these tests must meet the minimum prescribed values specified in Table 1. If the test results, depending on the state and the type of the line (restored or existing and Type I or Type II¹), are above the value intervals specified in Table 1, then the marking meets the requirements. If the values are lower than minimum that the marking does not comply with the requirements. If the retroreflection values are in tolerance interval, specified in the Table 1., a second stage of evaluation must be performed.

The second stage comprises of an additional 15 test measuring points. The relevant value represents the mean value of all the test points in the first stage and in the second stage of evaluation. If the relevant value is equal to or higher than the minimum requirement specified in Table 1, then the road marking is acceptable. On the other hand, marking does not comply with the quality standard and must be renewed.

### Table 1. Minimum values of retroreflection for restored and existing lines type I and II

<table>
<thead>
<tr>
<th>VISIBILITY AND STATE OF PAVEMENT</th>
<th>MINIMUM VALUE</th>
<th>INTERVAL (mcd·m⁻²·lx⁻¹)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>RESTORED LINES TYPE I</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Night-time visibility, dry pavement</td>
<td>RL ≥ 200</td>
<td>180 ≤ RL ≤ 220</td>
</tr>
<tr>
<td>Daytime visibility, dry pavement</td>
<td>Qd ≥ 130</td>
<td>110 ≤ Qd ≤ 150</td>
</tr>
<tr>
<td><strong>RESTORED LINES TYPE II</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Night time visibility, dry pavement</td>
<td>RL ≥ 300</td>
<td>270 ≤ RL ≤ 330</td>
</tr>
<tr>
<td>Daytime visibility, dry pavement</td>
<td>Qd ≥ 160</td>
<td>140 ≤ Qd ≤ 180</td>
</tr>
<tr>
<td><strong>EXISTING LINES TYPE I</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Night time visibility, dry pavement</td>
<td>RL ≥ 100</td>
<td>90 ≤ RL ≤ 110</td>
</tr>
<tr>
<td>Daytime visibility, dry pavement</td>
<td>Qd ≥ 100</td>
<td>90 ≤ Qd ≤ 110</td>
</tr>
<tr>
<td><strong>EXISTING LINES TYPE II</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Night time visibility, dry pavement</td>
<td>RL ≥ 150</td>
<td>130 ≤ RL ≤ 170</td>
</tr>
<tr>
<td>Daytime visibility, dry pavement</td>
<td>Qd ≥ 130</td>
<td>110 ≤ Qd ≤ 150</td>
</tr>
</tbody>
</table>

(Guidelines and technical requirements for performing works on renewing road markings, 2010)

The measurement of road markings retroreflectivity can be conducted in two ways:

- using the static method (daytime and night-time visibility);
- using the dynamic method (night-time visibility).

¹ Restored lines are measured between 30 and 60 days from the application, while existing lines are measured after winter. Type I lines are standard thin markings, while Type II are markings with enhanced performance in wet conditions.
2.1 STATIC METHOD FOR TESTING THE QUALITY OF ROAD MARKINGS

The static tests of daytime and night-time visibility of road markings are performed with hand-held retroreflectometers according to the European standard EN 1436:2009 - Road Marking Materials. Road Marking Performance for Road Users. Daytime visibility (Qd) is expressed and measured in \( \text{mcd} \cdot \text{m}^{-2} \cdot \text{lx}^{-1} \), observed at an angle of 2.29° at a distance of 30 m and it represents the value of the diffuse scattered light received by the observer. Night-time visibility or retroreflection (RL) measured in \( \text{mcd} \cdot \text{m}^{-2} \cdot \text{lx}^{-1} \), represents the retroreflection of a light beam from the tested surface at an angle of 2.29°, with a light inlet angle of 1.24° and at a distance of 30 m with low-beam headlights on a vehicle.

The standard EN 1436:2009 describes the basic measuring equipment, standard measuring condition of measuring equipment, practical applications and calibration of measuring equipment, uncertainty of measurement and conditions of wetness during rain in which night-time visibility of road markings is measured but it does not prescribe detailed methodology of retroreflection measurement. In other words, it does not define how many sections of a road should be measured, the length of these sections and the number of measuring points.

For this reason, the static tests in Croatia until 2010. where conducted according to Kentucky method and from 2010. are based on the German ZTV M02 method, which provides a detailed methodology for conducting the measurement.

![Figure 1. Device for measuring retroreflection of road markings](Fiolić et. al. 2012)

The main disadvantage of the Kentucky method (measurements are performed only in the first third of the length of the section in which road markings where done by one team in one day) is overcome with the German ZTV M02 method. According to ZTV M02, the scope of testing depends on the daily performance of the working team that applied the markings as shown in Table 2.

<table>
<thead>
<tr>
<th>Length of longitudinal markings applied in one day, km</th>
<th>The length of the other markings done in one day (m2)</th>
<th>Number of testing sections</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 1</td>
<td>&lt; 120</td>
<td>1</td>
</tr>
<tr>
<td>1 to 5</td>
<td>120 - 600</td>
<td>2</td>
</tr>
<tr>
<td>&gt; 5 to 10</td>
<td>&gt; 600 - 1200</td>
<td>3</td>
</tr>
<tr>
<td>&gt; 10</td>
<td>&gt; 1200</td>
<td>4</td>
</tr>
</tbody>
</table>

(Forschungsgesellschaft für strassen und verkehrswesen, 2002)

The length of the testing sections depends on the type of the line, meaning that for the continuous longitudinal lines the length is 100 m long and for intermittent longitudinal lines testing section comprises of 10 intermittent line.

Testing sections are selected according to the principle of randomness, with five measuring points being selected within each of them. For continuous longitudinal lines, measurement points are distributed at 100 m in length at equal intervals (start, 25 m, 50 m, 75 m and 100 m), while for intermittent longitudinal markings the measuring points are allocated in the middle of every other line. Figure 2 represents the methodology for measuring continuous longitudinal lines.
2.2 DYNAMIC METHOD FOR TESTING THE QUALITY OF ROAD MARKINGS

The dynamic method for testing retroreflection measures night-time visibility of the road markings with a dynamic measuring device along their entire length. The dynamic measuring device is placed on the right or left side of the vehicle depending on the position of road marking (Figure 3). The measuring process includes a vehicle driving on the road and reading the retroreflection coefficient of road markings. The length of the measuring interval in which the device will measure the mean values of a certain measuring section (25 m, 50 m or 100 m) is selected before the measurement. The greatest advantage of this method is that it tests road markings in their entire length which enables road authorities to have a complete and detailed insight in the quality of the markings.

Figure 3. Measuring vehicle of the Department of Traffic Signalling with the dynamic retroreflectometer (Fiolić et. al. 2012)

3 METHODS FOR MEASURING THE QUALITY OF TRAFFIC SIGNS

Retroreflection of traffic signs can be measured using handheld or mobile (dynamic) retroreflectometer. Mobile retroreflectometers are highly advanced, automated and to some extent a new technology which is still insufficiently tested. System is equipped with high sensitivity cameras installed on the measuring vehicle which measures the luminance while driving (Carlson 2011).

More reliable and precise way is by using handheld retroreflectometers whose geometry should match the values of the European Standard (EN 12899-1: Fixed, vertical road traffic signs - Part 1: Permanent signs) which implies an observation angle of 0.33° and an entrance angle of 5°.

The entrance angle is primarily determined by the position of the sign on the side of the road and the geometry of an oncoming vehicle position. It represents the angle that is formed between the light rays falling on the surface of the sign and the line that goes vertically from the surface. The observation angle is the angle...
between the incoming light ray and the reflected ray (EN 12899-1: Fixed, vertical road traffic signs - Part 1: Permanent signs) as shown in Figure 4.

![Figure 4. Entrance and observation angles for traffic sign](Šćukanec, A. et al., 2013.)

During the quality check, retroreflection of each colour on the traffic signs should be measured and comply the minimum coefficient of retroreflection (RA) prescribe in EN 12899-1 in order for the sign to be technically valid. According to EN 12899-1, the minimum coefficient of retroreflection RA (cd•lx⁻¹•m⁻²) of traffic signs must match the values shown in Table 3, Table 4 and Table 5. The coefficient of retroreflection (Rₐ) of all printed colours, except white, should not be less than 70 % of the values shown in these Tables.

### Table 3. Retroreflection coefficient Rₐ: Class Rₐ1 (cd•m⁻²•lx⁻¹)

<table>
<thead>
<tr>
<th>Geometry of measurements</th>
<th>Colour</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>α</td>
<td>β₁ (β₂=0)</td>
<td>white</td>
<td>yellow</td>
<td>red</td>
<td>green</td>
<td>blue</td>
<td>brown</td>
<td>orange</td>
<td>grey</td>
<td></td>
<td></td>
</tr>
<tr>
<td>12°</td>
<td>+5°</td>
<td>70</td>
<td>50</td>
<td>14.5</td>
<td>9</td>
<td>4</td>
<td>1</td>
<td>25</td>
<td>42</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>+30°</td>
<td>30</td>
<td>22</td>
<td>6</td>
<td>3.5</td>
<td>1.7</td>
<td>0.3</td>
<td>10</td>
<td>18</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>+40°</td>
<td>10</td>
<td>7</td>
<td>2</td>
<td>1.5</td>
<td>0.5</td>
<td>#</td>
<td>2.2</td>
<td>6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20°</td>
<td>+5°</td>
<td>50</td>
<td>35</td>
<td>10</td>
<td>7</td>
<td>2</td>
<td>0.6</td>
<td>20</td>
<td>30</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>+30°</td>
<td>24</td>
<td>16</td>
<td>4</td>
<td>3</td>
<td>1</td>
<td>0.2</td>
<td>8</td>
<td>14.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>+40°</td>
<td>9</td>
<td>6</td>
<td>1.8</td>
<td>1.2</td>
<td>#</td>
<td>#</td>
<td>2.2</td>
<td>5.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2°</td>
<td>+5°</td>
<td>5</td>
<td>3</td>
<td>1</td>
<td>0.5</td>
<td>#</td>
<td>#</td>
<td>1.2</td>
<td>3</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>+30°</td>
<td>2.5</td>
<td>1.5</td>
<td>0.5</td>
<td>0.3</td>
<td>#</td>
<td>#</td>
<td>0.5</td>
<td>1.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>+40°</td>
<td>1.5</td>
<td>1.0</td>
<td>0.5</td>
<td>0.2</td>
<td>#</td>
<td>#</td>
<td>1.2</td>
<td>0.9</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Indicates "Value greater than zero but not significant or applicable".

(EN 12899-1: Fixed, vertical road traffic signs - Part 1: Permanent signs)

### Table 4. Retroreflection coefficient Rₐ: Class Rₐ2 (cd•m⁻²•lx⁻¹)

<table>
<thead>
<tr>
<th>Geometry of measurements</th>
<th>Colour</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>α</td>
<td>β₁ (β₂=0)</td>
<td>white</td>
<td>yellow</td>
<td>red</td>
<td>green</td>
<td>dark green</td>
<td>blue</td>
<td>brown</td>
<td>orange</td>
<td>gray</td>
<td></td>
</tr>
<tr>
<td>12°</td>
<td>+5°</td>
<td>250</td>
<td>170</td>
<td>45</td>
<td>45</td>
<td>20</td>
<td>20</td>
<td>12</td>
<td>100</td>
<td>125</td>
<td></td>
</tr>
<tr>
<td></td>
<td>+30°</td>
<td>150</td>
<td>100</td>
<td>25</td>
<td>25</td>
<td>15</td>
<td>11</td>
<td>8.5</td>
<td>60</td>
<td>75</td>
<td></td>
</tr>
<tr>
<td></td>
<td>+40°</td>
<td>110</td>
<td>70</td>
<td>15</td>
<td>12</td>
<td>6</td>
<td>8.5</td>
<td>6.0</td>
<td>29</td>
<td>55</td>
<td></td>
</tr>
<tr>
<td>20°</td>
<td>+5°</td>
<td>180</td>
<td>120</td>
<td>25</td>
<td>21</td>
<td>14</td>
<td>14</td>
<td>8</td>
<td>65</td>
<td>90</td>
<td></td>
</tr>
<tr>
<td></td>
<td>+30°</td>
<td>100</td>
<td>70</td>
<td>14</td>
<td>12</td>
<td>11</td>
<td>8</td>
<td>5.0</td>
<td>40</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td></td>
<td>+40°</td>
<td>95</td>
<td>60</td>
<td>13</td>
<td>11</td>
<td>5</td>
<td>7</td>
<td>3</td>
<td>20</td>
<td>47</td>
<td></td>
</tr>
<tr>
<td>2°</td>
<td>+5°</td>
<td>5</td>
<td>3</td>
<td>1</td>
<td>0.5</td>
<td>0.5</td>
<td>0.2</td>
<td>0.2</td>
<td>1.5</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
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<td>2.5</td>
<td>1.5</td>
<td>0.4</td>
<td>0.3</td>
<td>0.3</td>
<td>#</td>
<td>#</td>
<td>1</td>
<td>1.2</td>
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</tr>
<tr>
<td></td>
<td>+40°</td>
<td>1.5</td>
<td>1.0</td>
<td>0.3</td>
<td>0.2</td>
<td>0.2</td>
<td>#</td>
<td>#</td>
<td>1</td>
<td>0.7</td>
<td></td>
</tr>
</tbody>
</table>

* Indicates "Value greater than zero but not significant or applicable".

(EN 12899-1: Fixed, vertical road traffic signs - Part 1: Permanent signs)
Table 5. Retroreflection coefficient $R_A$: Class $R_A3$ \( (\text{cd} \cdot \text{m}^{-2} \cdot \text{lx}^{-1}) \)

<table>
<thead>
<tr>
<th>Geometry of measurements</th>
<th>Colour</th>
<th>white</th>
<th>yellow</th>
<th>red</th>
<th>green</th>
<th>blue</th>
<th>orange</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha$</td>
<td>$\beta_1$ ($\beta_2=0$)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.1°</td>
<td>+5°</td>
<td>850</td>
<td>550</td>
<td>170</td>
<td>85</td>
<td>55</td>
<td>260</td>
</tr>
<tr>
<td></td>
<td>+20°</td>
<td>600</td>
<td>390</td>
<td>120</td>
<td>60</td>
<td>40</td>
<td>130</td>
</tr>
<tr>
<td></td>
<td>+30°</td>
<td>425</td>
<td>275</td>
<td>85</td>
<td>40</td>
<td>28</td>
<td>95</td>
</tr>
<tr>
<td>0.2°</td>
<td>+5°</td>
<td>625</td>
<td>400</td>
<td>125</td>
<td>60</td>
<td>40</td>
<td>140</td>
</tr>
<tr>
<td></td>
<td>+20°</td>
<td>450</td>
<td>290</td>
<td>90</td>
<td>45</td>
<td>30</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>+30°</td>
<td>325</td>
<td>210</td>
<td>65</td>
<td>30</td>
<td>20</td>
<td>70</td>
</tr>
<tr>
<td>0.33°</td>
<td>+5°</td>
<td>425</td>
<td>275</td>
<td>85</td>
<td>40</td>
<td>28</td>
<td>95</td>
</tr>
<tr>
<td></td>
<td>+20°</td>
<td>300</td>
<td>195</td>
<td>60</td>
<td>30</td>
<td>20</td>
<td>65</td>
</tr>
<tr>
<td></td>
<td>+30°</td>
<td>225</td>
<td>145</td>
<td>45</td>
<td>20</td>
<td>15</td>
<td>49</td>
</tr>
</tbody>
</table>

(Consiglionazionedell'ericerche: Common understanding of assessment procedure (CUAP): Microprismatic retro-reflective sheetings, 2002)

4 TESTING THE QUALITY OF TRAFFIC SIGN AND ROAD MARKINGS IN CROATIA

Quality testing of road markings and traffic signs, as mentioned before, provides road authorities with an inside look at how road markings and traffic signs perform, in which condition they are and how well are conductors doing their job.

In the Croatia, quality testing’s of road markings and traffic sign are conducted by the Department of Traffic Signalling at the Faculty of Transport and Traffic Sciences, University of Zagreb for more than ten years. In next sections, an overview of these measurements will be presented.

4.1 QUALITY TESTING OF ROAD MARKINGS IN CROATIA

From the year 2000, the Department of Traffic Signalling is conducting measurements of retroreflection using static measuring method on the roads across Croatia. From the 2010 until today, majority of measurements was done for Croatian Roads ltd. using dynamic retroreflectometer Zehntner ZDR 6020. These measurements were part of the long-term quality check plan in Croatia that was divided in four parts as shown in the Figure 5.

Figure 5. Total number of measured kilometres of longitudinal road markings on state roads in Croatia

(Prepared by the authors)

From the above, it can be concluded that in seven-year period (from 2010 until 2017) on the state roads in Croatia a total of 68 713 km of longitudinal road markings was measured. From this total, 44 262 km of markings were new markings which were measured as a part of conductor’s quality control plan made from Croatian Roads ltd. Other 24 451 km of measured markings were existing markings which were measured for the purpose of checking their compliancy with the minimal prescribed values (from the Table 1) and creating a maintenance plan in order to achieve desired visibility with optimal costs of renewal.
During this control plan period, a systematic increase of retroreflection, i.e. quality of road markings on the state roads has been recorded as shown in Figure 7. From the 2010 until 2017 an average retroreflection of road markings on state roads in Croatia has increased for 5% from 262.6 mcd m⁻² lx⁻¹ to 275.5 mcd m⁻² lx⁻¹. In should be noted that majority (more than 90%) of road markings in Croatia are made from solventborne paint which is, from the durability and visibility point of view, the poorest material. Despite that, overall quality of road markings on the state roads in Croatia is satisfying even though road markings in almost every Croatian county were maintained by different conductor.

These results show that systematic road markings control plan can lead to the increase in their quality and thus results in more efficient maintenance system and improved visibility which ultimately leads to safer roads.

The Department is involved in static measurement of reflectivity of road markings on state roads in the Republic of Croatia since 2003. In that period, the Department has continuously (every year) measured retro-reflection of road markings on most of the state roads providing a valuable source of data. However, measurement was not performed in the same exact locations each year, making year on year comparisons difficult. This was due to road maintenance plans and the change in measuring methods in year 2010. Analysis of the data revealed that there was a total 6 roads where all measurements were conducted in exactly the same places from 2003 till 2013. In Table 6 is average measuring results of night visibility (RL) for each year. Table 6 present the road classification and number (according Croatian laws), type of line (ML – middle line, SLR – side line right, SLL – side line left) and values of night visibility (RL) for each year. The last column (TG) represent the total growth of night for each line on each row in the period from 2003 until 2013.

Table 6. Measured values of average night visibility (RL) on selected roads in Republic of Croatia per each year (2003. – 2013.)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>D36</td>
<td>ML</td>
<td>187.95</td>
<td>234.80</td>
<td>252.68</td>
<td>286.62</td>
<td>310.57</td>
<td>334.40</td>
<td>358.31</td>
<td>382.22</td>
<td>406.14</td>
<td>430.07</td>
<td>454.00</td>
<td>21.51%</td>
</tr>
<tr>
<td></td>
<td>SLR</td>
<td>187.95</td>
<td>234.80</td>
<td>252.68</td>
<td>286.62</td>
<td>310.57</td>
<td>334.40</td>
<td>358.31</td>
<td>382.22</td>
<td>406.14</td>
<td>430.07</td>
<td>454.00</td>
<td>21.51%</td>
</tr>
<tr>
<td></td>
<td>SLL</td>
<td>187.95</td>
<td>234.80</td>
<td>252.68</td>
<td>286.62</td>
<td>310.57</td>
<td>334.40</td>
<td>358.31</td>
<td>382.22</td>
<td>406.14</td>
<td>430.07</td>
<td>454.00</td>
<td>21.51%</td>
</tr>
<tr>
<td>D30</td>
<td>ML</td>
<td>142.03</td>
<td>222.94</td>
<td>242.26</td>
<td>252.32</td>
<td>260.66</td>
<td>271.45</td>
<td>272.35</td>
<td>285.15</td>
<td>290.18</td>
<td>305.12</td>
<td>308.07</td>
<td>116.90%</td>
</tr>
<tr>
<td></td>
<td>SLR</td>
<td>189.00</td>
<td>204.30</td>
<td>236.63</td>
<td>246.53</td>
<td>253.15</td>
<td>260.37</td>
<td>263.68</td>
<td>268.40</td>
<td>286.40</td>
<td>288.07</td>
<td>290.43</td>
<td>53.67%</td>
</tr>
<tr>
<td></td>
<td>SLL</td>
<td>164.00</td>
<td>234.93</td>
<td>241.46</td>
<td>248.43</td>
<td>250.55</td>
<td>259.64</td>
<td>262.00</td>
<td>266.69</td>
<td>281.00</td>
<td>287.64</td>
<td>290.03</td>
<td>76.24%</td>
</tr>
<tr>
<td>D37</td>
<td>ML</td>
<td>188.51</td>
<td>219.06</td>
<td>227.60</td>
<td>288.80</td>
<td>290.40</td>
<td>294.55</td>
<td>299.54</td>
<td>307.35</td>
<td>313.29</td>
<td>309.56</td>
<td>317.87</td>
<td>68.62%</td>
</tr>
<tr>
<td></td>
<td>SLR</td>
<td>220.15</td>
<td>262.30</td>
<td>279.83</td>
<td>262.76</td>
<td>266.54</td>
<td>249.77</td>
<td>271.24</td>
<td>278.65</td>
<td>285.11</td>
<td>288.04</td>
<td>289.77</td>
<td>31.62%</td>
</tr>
<tr>
<td></td>
<td>SLL</td>
<td>238.67</td>
<td>255.03</td>
<td>263.60</td>
<td>306.35</td>
<td>327.45</td>
<td>332.06</td>
<td>278.31</td>
<td>285.41</td>
<td>293.11</td>
<td>290.17</td>
<td>293.78</td>
<td>23.09%</td>
</tr>
<tr>
<td>D2</td>
<td>ML</td>
<td>190.47</td>
<td>220.76</td>
<td>247.15</td>
<td>292.75</td>
<td>278.69</td>
<td>269.61</td>
<td>280.57</td>
<td>287.44</td>
<td>269.15</td>
<td>293.07</td>
<td>298.43</td>
<td>56.68%</td>
</tr>
</tbody>
</table>
From the collected data it can be concluded that the middle line has the highest growth of night visibility. As the middle line is the most important to the driver’s orientation, the growth of almost 70% is satisfying. Figure 7 shows the comparison and growth of night for each line on each road in the period 2003 – 2013.

In order to explore influence of the quality of road markings on the traffic safety the statistical analysis, i.e. correlation coefficient between traffic accidents and retroreflection of road markings was done. For the analysis the retroreflection for each road (except road DC37) and each year was taken as an average of values of MS, SLR and SLL from table 6. Road DC37 was excluded from the analysis because it is a road with low average daily annual traffic volume but more importantly the number of accidents represented extreme values (low number of accidents) in analysis. The data related to the number of traffic accidents was obtained by the police authorities relevant to the area in which the road is for the period of 2004 till 2013 as shown in table 7.

Table 7. Number of traffic accidents by year and average retroreflection values

<table>
<thead>
<tr>
<th>Year</th>
<th>Road</th>
<th>DC36</th>
<th>DC30</th>
<th>DC7</th>
<th>DC2</th>
<th>DC55</th>
</tr>
</thead>
<tbody>
<tr>
<td>2004</td>
<td>Accidents</td>
<td>48</td>
<td>51</td>
<td>184</td>
<td>284</td>
<td>121</td>
</tr>
<tr>
<td></td>
<td>R&lt;sub&gt;L&lt;/sub&gt;</td>
<td>220.72</td>
<td>241.75</td>
<td>245.46</td>
<td>234.69</td>
<td>214.91</td>
</tr>
<tr>
<td>2005</td>
<td>Accidents</td>
<td>49</td>
<td>41</td>
<td>180</td>
<td>252</td>
<td>105</td>
</tr>
<tr>
<td></td>
<td>R&lt;sub&gt;L&lt;/sub&gt;</td>
<td>240.12</td>
<td>249.81</td>
<td>257.01</td>
<td>269.42</td>
<td>239.00</td>
</tr>
<tr>
<td>2006</td>
<td>Accidents</td>
<td>49</td>
<td>38</td>
<td>171</td>
<td>239</td>
<td>103</td>
</tr>
<tr>
<td></td>
<td>R&lt;sub&gt;L&lt;/sub&gt;</td>
<td>249.09</td>
<td>256.75</td>
<td>285.97</td>
<td>279.33</td>
<td>241.06</td>
</tr>
<tr>
<td>2007</td>
<td>Accidents</td>
<td>47</td>
<td>21</td>
<td>165</td>
<td>212</td>
<td>98</td>
</tr>
<tr>
<td></td>
<td>R&lt;sub&gt;L&lt;/sub&gt;</td>
<td>254.79</td>
<td>267.35</td>
<td>276.46</td>
<td>276.53</td>
<td>249.60</td>
</tr>
<tr>
<td>2008</td>
<td>Accidents</td>
<td>51</td>
<td>32</td>
<td>176</td>
<td>218</td>
<td>94</td>
</tr>
<tr>
<td></td>
<td>R&lt;sub&gt;L&lt;/sub&gt;</td>
<td>263.82</td>
<td>271.77</td>
<td>258.79</td>
<td>258.54</td>
<td>242.80</td>
</tr>
<tr>
<td>2009</td>
<td>Accidents</td>
<td>39</td>
<td>22</td>
<td>132</td>
<td>188</td>
<td>90</td>
</tr>
<tr>
<td></td>
<td>R&lt;sub&gt;L&lt;/sub&gt;</td>
<td>266.01</td>
<td>278.28</td>
<td>283.03</td>
<td>270.41</td>
<td>259.55</td>
</tr>
</tbody>
</table>
To do a correlation analysis, first it is needed to check if the data is normally distributed. For that purposes the Kolmogorov-Smirnov test was done which showed that the retroreflection data (R_L) is normally distributed but the Accidents data is not normally distributed. Because of the not normal distribution of accidents data for the correlation analysis nonparametric Spearman test was used. The results of the test showed that the Spearman’s correlation between retroreflection of road markings and traffic accidents is -0.201 as shown in Table 8. The results show that the correlation between retroreflection and traffic accidents is relatively week and negative meaning that with the increase of retroreflection will cause the decrease of traffic accidents.

Table 8. Results of the Spearman correlation test

<table>
<thead>
<tr>
<th></th>
<th>Accidents</th>
<th>R_L</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Spearman’s rho</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Correlation</td>
<td>1.000</td>
<td>-0.201</td>
</tr>
<tr>
<td>Sig. (2-tailed)</td>
<td>.</td>
<td>.161</td>
</tr>
<tr>
<td>N</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td><strong>R_L</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Correlation</td>
<td>-0.201</td>
<td>1.000</td>
</tr>
<tr>
<td>Sig. (2-tailed)</td>
<td>.161</td>
<td>.</td>
</tr>
<tr>
<td>N</td>
<td>50</td>
<td>50</td>
</tr>
</tbody>
</table>

From the similar previous studies it is clear that it is complicated to determine the correlation between the road markings retro-reflection and night-time safety due to the impact of many factors related to driver and constructional features of the road. Although some studies (Dravitzki et al., 2006; Bahar et. al., 2006) failed to determine this correlation, several newer studies (Avelar et al., 2014; Smadi et al., 2008) like this one have shown a statistical association between road marking retro-reflectivity and night-time safety. Although, the correlation in this study is week one can conclude that overall road safety is influenced by road markings’ visibility level. From all the above it can be concluded that the quality and visibility of road markings ensure clear trajectories of the driving path, remove a substantial workload off the driver and provide the anticipatory stimuli of the road environment. This is for sure proof that the increase in general traffic safety on the roads can be achieved with systematic retro-reflection testing of road markings which, as results of this study show, will affect the conclusive increased road markings’ night and daytime visibility.

4.2 QUALITY TESTING OF TRAFFIC SIGNS IN CROATIA

As a basic means of communication between the road authorities and traffic participants, traffic signs with the use of colours, shapes and symbols give important information with which they manage, regulate, inform and warn road users to ensure their safe movement throughout the transport network (Fleyeh and Roch, 2013). To efficiently perform their functions, traffic signs should be easily recognizable and locatable within a complex visual scene, properly maintenance so they could clearly indicate the status of the message (legal, warning or information) and convey the message efficiently so that the driver has sufficient time to act accordingly (Jamson et al. 2005).

This is especially important in the conditions of reduced visibility in which traffic signs provide crucial information related to the condition in front. Improper installation and maintenance of road signs can significantly affect their perception, the quality of the transmitted message that they carry, thus affecting drivers’ reaction time and general road safety.

To ensure an adequate visibility of traffic signs measurements of retroreflection should be periodically tested. In Croatia, these measurements are conducted by the Department of Traffic Signalling at the Faculty of
Transport and Traffic Sciences, University of Zagreb. During two-year period, from 2015 to 2017, Department has measured retroreflection on all of traffic signs on a state roads in Croatia as a part of control plan by Croatian Roads, ltd. This means that the total amount of 149,435 traffic signs was measured on the 6,957.23 km of state roads network in Croatia. Except the retroreflection properties on each of 149,435 traffic signs, additional information about traffic signs was also collected for the purposes of creating traffic signs database. These additional information’s are related to the following:

- Chainage (road kilometre position of sign)
- GPS coordinate of the sign
- Traffic signs code according to the Croatian legislation
- Direction of sign
- Class of retroreflection material
- Dimensions
- Shape
- Name of the manufacturer
- Year of manufacture
- Mounting characteristics and plate thickness
- Height of traffic signs
- Distance from the edge of the pavement and
- Digital photography of traffic sign.

From a total of 149,435 traffic signs which are located on the Croatian state roads network, the most common are information signs (28.02%) and mandatory signs (25.82%) as shown in Figure 8.

![Figure 8. Percentage of traffic signs by type on state roads in Croatia (Prepared by the authors)](image)

The analysis of the retroreflective material from which traffic signs are made, showed that 64.92% of all signs are made from Class I, 27.81% from Class II and 7.27% from Class III material (Figure 9). Because Croatian regulations demand only several signs to be made from Class II and Class III material majority of the signs are made from Class I which is the material with lowest retroreflective properties.
More detailed analysis of the traffic signs quality shows that overall state of the signs is satisfying. From a total of 149,435 traffic signs, 71.12% of them meet the technical requirements (retroreflection properties and technical quality\(^2\)) and 28.88% do not meet (Figure 10).

The main reason why this 28.88% of the signs do not meet technical requirements is their age. From the Figure 11, one can conclude that average age of the signs that do not meet technical requirements made from Class I is 12.7, and Class II 10.1 which is more than the producers guaranty. Average age of the technically not good signs made from Class III is 8.6 years. The reason for that is that Class III materials according to Croatian regulations are used for type of chevron signs which are usually positioned lower that other traffic signs and closer to the road which is why they are more often damage by small stones.

\(^2\) Traffic signs is technically correct when he is according to the valid Croatian regulations and is not damaged in some ways.
All collected data represents an important knowledge for the road authorities in this case Croatian Roads, ltd. based on which a complete plan of maintenance program may be created in order to achieve efficient traffic signs and improve road safety while optimizing costs.

5 CONCLUSION

Road markings and traffic signs represent an important part of road infrastructure which provides road users with an information’s related to the traffic situation in front. In road traffic, the impact of direct information is more pronounced than in other modes of traffic due to the large number of participants, the intensity of traffic flows and the passing of decision-making of individual participants in different situations.

To ensure the proper and in-time transmission of the information which road markings and traffic signs carry, they should be properly placed and maintenance. Important element of the maintenance process is testing their quality which is mainly related to the measurement of retroreflection. Properly installed and maintenance road markings and traffic signs are especially important in conditions of reduced visibility (night, dawn, fog, rain etc.), in which road users have a limited capability to use peripheral vision, depth perception and colour vision.

From the analysis presented in this paper it can be concluded that the overall quality of traffic sings and road markings in Croatia is satisfying. Also, the continuous measurements on the state roads in Croatia, conducted by the Department of Traffic Signalling at the Faculty of Transport and Traffic Sciences, University of Zagreb, prove that the quality of traffic signs and road markings may be maintained on the satisfactory level and even increased. Using modern equipment and methods detailed insight into a quality of road markings and traffic signs is provided based on which road authorities may develop full-scale maintenance program focused on improving traffic safety and optimizing maintenance costs.

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World Health Organization (Available at http://www.who.int/mediacentre/factsheets/fs358/en/)
## PAPER TITLE
Derivation of transition probability matrices for damage assessment in road networks: a Colombian case study

## TRACK
5- Sustainable Pavement Design & Management

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<th>COUNTRY</th>
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<td>Fuentes, Luis G.</td>
<td>Head of Department of Civil and Environmental Engineering</td>
<td>Universidad del Norte</td>
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## KEYWORDS:
Pavement performance model, Transition Probability Matrix, Deterioration, PMS, Pavement

## ABSTRACT:
A Pavement Management System (PMS) provides the tools to efficiently aid the decision-making process of a highway agency. Pavement performance models can be used on a PMS to forecast the condition of a pavement section, hence, one could select the appropriate maintenance and rehabilitation action in order to reduce road agency and road user costs. This investigation focuses on the development of stochastic models for predicting pavement deterioration throughout the Colombian territory, encompassing the various levels of temperature and traffic encountered in its territory. The analysis comprised nine transition probability matrices corresponding to the interaction of three levels (high, medium and low) defined for both temperature and traffic for a total of 72 evaluated pavement sections. Additionally, as a contribution of this work, a methodology for identifying the criticality of transition probability matrices was developed by means of a criticality index. The main conclusions are related to the importance of transition probability matrices to predict pavement deterioration and the value of making use of these in the context of local pavement management systems to ease an appropriate and timely pavement management. Furthermore, the use of the results from this research will allow for optimizing available resources by means of the prediction of future road conditions and efficient decision-making.
1 INTRODUCTION

Ensuring pavement adequate performance as well as security and comfort are highly relevant aspects in the modern world when considering the vast amount of goods and services transported in the road networks and substantial profit transferred to socio-economic development. As a result, pavements constitute a vital element in transport infrastructure that needs maximum optimization, adequate use, maintenance and rehabilitation, which can be summarized into appropriate Pavement Management System (PMS) practices. For this reason, local and national authorities should conduct adequate and efficient practices for managing road infrastructure that guarantees adequate levels of comfort, providing suitable performance and durability of the pavements in order to facilitate traffic circulation, minimization of reinvestment costs and other social and ecological costs associated to their use over the design period. In this context, tools and methodologies for assessing the current and future condition of pavements are required for assisting managers and decision-makers in the prioritization of resources and allocation of these into maintenance & rehabilitation activities in the road networks comprised into a PMS. One of these tools are pavement deterioration models that allow for, among other aspects to:

- Predict the condition of a given pavement section at any period in time.
- Monitor the evolution of deterioration over time and have constant follow-up of the impact of traffic regimes and environmental conditions imposed on the pavement structure.
- Prioritize projects and make rational and efficient use of available resources.

In consistency with these aspects, the National Road Institute (INVIAS) created a program defined as Administradores Viales (Road Administrators) intended for assessing the national road infrastructure by using two main methodologies: (i) Technical Criterion and, (ii) Visual Criterion. These two methodologies are used for diagnosing pavement condition through reliable measurements and representative data that serves for decision-making processes and resources allocation for maintaining the road network in good condition over time. The program’s main objective is keeping the road infrastructure in good condition to foster the Nations’ socioeconomic development, through appropriate maintenance and rehabilitation when required. More specifically, the pavement condition is assessed by means of functional and structural indicators, which accounts for an inventory of distresses and surface damages, as part of the Technical Criterion. The distresses include: potholes, cracking, rutting, and raveling, among others (INVIAS, 2008).

The present investigation aims to develop probabilistic models to determine the deterioration of flexible pavements in Colombian roads based upon data from INVIAS. These models could be used by local and national authorities in the decision-making process to ensure a rational prioritizations of rehabilitation projects in their respective PMS. The methodology adopted for this analysis considers the two main factors involved in pavement deterioration: temperature and traffic.

2 PAVEMENT MANAGEMENT SYSTEMS

Pavement management systems are essential tools for prioritizing investments in order to maximize benefits and minimize the costs associated to maintenance and rehabilitation of a given road network. The purpose of a PMS is maintaining the road network in such condition that it allows for an adequate level of serviceability along the life cycle of the pavement. More specifically, the life cycle involves several stages: planning, design, construction, management and conservation. In addition, performance assessments as well as damage models are vital elements of a PMS since
these dictate, to a great extent, the activities required to maintain, preserve and rehabilitate the pavement along its lifecycle. In this regard, greater accuracy in these models will have a beneficial impact on the cost saving for a given project, allowing for a maximization of resources and efficient decision-making strategies (Haas, 2003).

3 PAVEMENT DETERIORATION MODELS

Assuming that optimum materials are used and suitable methods are applied during the construction of a pavement structure, the deterioration to which it will be subjected throughout its life cycle is mainly due to traffic load and environmental conditions. Thus, as time goes by, pavement performance will suffer from accumulation of strain levels and environmental conditions, resulting in deterioration of the surface layer (e.g., rutting, cracking, potholes, etc.), reducing the serviceability level over time and its capacity to supply for the demand of vehicles. According to Haas, 2003, deterioration or damage models are typically classified into three (3) major groups depending on the modelling-approach.

1. Empirical Models: These models used statistical techniques to predict pavement deterioration using explanatory variables. The dependent variable is any pavement performance indicator of interest, and is usually related to one or more explanatory variables representing pavement age, traffic and environmental conditions (Prozzi, 2001). The applicability of the models is limited by the range of data with which these were estimated.

2. Mechanical-Empirical Models: These models use pavement responses (stress, strain and deflections) and material characteristics to determine pavement performance. Other variables such as accumulated traffic, pavement age and pavement damage can also be considered. Pavement response is correlated to pavement performance and finally calibrated to the actual conditions of the road.

3. Probabilistic Models: Probabilistic models can be used to estimate the probability that the pavement is in a certain state or condition. These models are usually described by Transition Probability Matrices (TPM). The derivation of TPM has been traditionally carried out using two methods: (1) Based on historical data, observing the way a road network deteriorates from one year to another, and use this to estimate the TPM, and (2) using a panel of experienced engineers to estimate the probabilities of the TPM based on experts opinion (Ortiz et al. 2006).

This research study utilized Markov chain models to forecast future pavement conditions. These were of Empirical-Probabilistic nature, which comprised developing Transition Probability Matrix (TPM) by using historic data of pavement condition of the Colombian road network. The reasons for the selection of this type of models were: (i) TPM is a simple-to-use methodology and easily adjusted to the qualitative nature of the data provided by INVIAS, (ii) probabilistic models are intended for predicting future global conditions of pavement sections at the network level, helping to allocate efforts and resources in an efficient manner for maintenance and rehabilitation, which, in essence, cannot be performed through mechanistic modeling and; (iii) this methodology captures the stochastic nature of pavement deterioration that is caused by numerous variables and determining the incidence of these individually would be impractical at the network level (Fuentes et al. 2014, Prozzi, J. A., and S. M. Madanat, 2004; Prozzi, Jorge A., and Feng Hong, 2008).

4 MARKOV THEORY APPLIED TO PAVEMENT PERFORMANCE

The Markov prediction model is a stochastic process governed by three conditions (Ortiz et al. 2006):

1. The process is discrete in time.
2. The process must have a countable or finite state space (condition states).
3. The process must satisfy the “Markov property”.

Different researchers have shown that Markov Chain process can be used in the determination of pavement deterioration (Fuentes et al. 2014, Lytton, 1987; Ortiz et al. 2006; Zheng, 2005; Wang, 1993; Nasseri et al. 2009).

The pavement condition can be modeled by stationary or nonstationary Markov chains. In the case of stationary Markov chains, it is considered that the pavement deteriorates following the behavior set by a single transition probability matrix independent of time. On the other hand, if the rate of deterioration of the pavement changes with time the process may be modeled by a nonstationary Markov chain.
The initial state of the process is defined by the vector: \( \mathbf{a}_0 = (a_1, a_2, \ldots, a_n) \). At the network level, the components of \( \mathbf{a}_0 \) indicate the current condition proportions of the network. To model pavement performance over time, one must set the Transition Probability Matrix (TPM), identified by \( \mathbf{M} \). The general form of \( \mathbf{M} \) is given by Equation 1.

\[
\mathbf{M} = \begin{bmatrix}
P_{11} & P_{12} & \cdots & P_{1j} \\ P_{21} & P_{22} & \cdots & P_{2j} \\ \vdots & \vdots & \ddots & \vdots \\ P_{i1} & P_{i2} & \cdots & P_{ij} \\
\end{bmatrix}
\]

(1)

Where

\[
\sum_{j=1}^{n} P_{ij} = 1 \quad \text{for} \quad i = 1, 2, \ldots, n
\]

(2)

The matrix defined in Equation (1) contains all of the information necessary to model the transition process among the condition states. The transition probabilities, \( P_{ij} \), indicate the probability of the portion of the network in condition state \( i \) moving to condition state \( j \) in one duty cycle. \( P_{ij} \) values must be nonnegative. Taking into account that pavements do not improve its condition without receiving a preservation or rehabilitation treatment, Equation (1) can be rewritten as follows:

\[
\mathbf{M} = \begin{bmatrix}
P_{11} & P_{12} & \cdots & P_{1j} \\ 0 & P_{22} & \cdots & P_{2j} \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & \cdots & 1 \\
\end{bmatrix}
\]

(3)

To determine \( P_{ij} \) values is necessary to monitor pavement deterioration and have access to historical data depicting changes on its state condition over time. When historical data is not available, these models can be developed based on expert judgment. The method for calculating \( P_{ij} \) values is as follows:

\[
P_{ij} = \frac{N_{ij}}{N_i}
\]

(4)

Where,

\( N_{ij} \): Refers to the number of pavement sections that shifted from \( i \) to \( j \) state condition in a period between \( t \) and \( t + 1 \).

\( N_i \): The number of pavement sections that began the period between \( t \) and \( t + 1 \) with a condition \( i \).

The initial vector size and TPM for this study are presented depicted on Equation (5) and (6) respectively.

\[
\mathbf{a}_0 = (a_{01}, a_{02}, a_{03}, a_{04}, a_{05})
\]

(5)

Where,

\( a_{0i} = N_i/N \) and \( N_i \) is the number of pavement sub-sections with a 1000-m length, which classifies as \( i \) state condition.

Finally, Equation (6) depicts the general TPM adopted for this research study.

\[
\mathbf{M} = \begin{bmatrix}
P_{11} & P_{12} & P_{13} & P_{14} & P_{15} \\ 0 & 0 & P_{23} & P_{24} & P_{25} \\ 0 & 0 & 0 & P_{34} & P_{35} \\ 0 & 0 & 0 & 0 & 1 \\
\end{bmatrix}
\]

(6)

Where,

\( P_{ij} = N_{ij}/N \) and \( N_i \) is the amount of changes or transitions from \( i \) to \( j \) condition. For instance, it could be related to a shift from a Good to a Fair state condition of the pavement. \( N_i \) is the summation of elements in the \( i \) row in matrix \( \mathbf{M} \).
5 TRAFFIC AND ENVIRONMENTAL CONDITIONS OF THE ROAD NETWORK

Data categorization was thoroughly conducted based on the information provided by INVIAS. Traffic information associated with the road network was gathered from the INVIAS database. The following thresholds were proposed to define different traffic levels: (1) Low ADT: less than 700 vehicles; (2) Medium ADT: between 700-2000 vehicles and; (3) High ADT: higher than 2000 vehicles. These levels can be used to uniformly describe the entire road network.

For climatic data categorization, historic records obtained from the Instituto de Hidrología, Meteorología y Estudios Ambientales de Colombia (IDEAM) were analyzed. The Annual-Mean Air Temperature associated with the evaluated pavement sections were used, which is consistent with the Shell methodology to define temperatures for pavement design (Shell, 1978). The corresponding thresholds for temperature are: (1) Low temperature category: below 20 °C; (2) Medium temperature category: between 20 °C and 27 °C and; (3) High temperature category: above 27 °C. Table 1 summarizes all the possible combinations of environmental and traffic conditions, and corresponding symbols considered in the following sections of the paper.

Table 1. Combinations of environmental and traffic scenarios

<table>
<thead>
<tr>
<th>Category</th>
<th>Symbol</th>
</tr>
</thead>
<tbody>
<tr>
<td>High Temperature and High ADT</td>
<td>[A,A]</td>
</tr>
<tr>
<td>Medium Temperature and High ADT</td>
<td>[M,A]</td>
</tr>
<tr>
<td>Low Temperature and High ADT</td>
<td>[B,A]</td>
</tr>
<tr>
<td>High Temperature and Medium ADT</td>
<td>[A,M]</td>
</tr>
<tr>
<td>Medium Temperature and Medium ADT</td>
<td>[M,M]</td>
</tr>
<tr>
<td>Low Temperature and Medium ADT</td>
<td>[B,M]</td>
</tr>
<tr>
<td>High Temperature and Low ADT</td>
<td>[A,B]</td>
</tr>
<tr>
<td>Medium Temperature and Low ADT</td>
<td>[M,B]</td>
</tr>
<tr>
<td>Low Temperature and Low ADT</td>
<td>[B,B]</td>
</tr>
</tbody>
</table>

It is important to mention that those pavement sections that had already undergone any rehabilitation or maintenance activity were excluded from the analysis in order to solely consider pavement sections affected by deterioration due to climatic conditions and traffic loading. Table 2 details all road sections included in this investigation associated with each environmental and traffic scenario.
### Table 2 Temperature and traffic scenarios of the road sections

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>High</th>
<th>Medium</th>
<th>Low</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temperature</td>
<td>Higher than 27°C</td>
<td>27°C - 20°C</td>
<td>Lower than 20°C</td>
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<tr>
<td>Road Section</td>
<td></td>
<td></td>
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<tr>
<td>Fundación-Aracataca (Magdalena)</td>
<td>Honda-rio ermitaño (Tolima)</td>
<td>Barrosa-tunja (Boyacá)</td>
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<td>Dabeiba-santa fe de Antioquia (Antioquia)</td>
<td>Belen-sacama (Boyacá)</td>
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<tr>
<td>El viajano-guayepo (Córdoba)</td>
<td>Turbo-chigorodo (Antioquia)</td>
<td>Duitama-la palmera (Boyacá)</td>
<td></td>
</tr>
<tr>
<td>Lorica-chinú (Córdoba)</td>
<td>Bucaramanga-san alberto (Santander)</td>
<td>Chiquinquira-saboya (Boyacá)</td>
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</tr>
<tr>
<td>Planeta rica-montería (Córdoba)</td>
<td>Puente nacional - barbosa (Santander)</td>
<td>Zipaquira-nemocon (Cundinamarca)</td>
<td></td>
</tr>
<tr>
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<td>Lerida-antiguo armero (Tolima)</td>
<td>Zipaquira-cogua (Cundinamarca)</td>
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</tr>
<tr>
<td>Cerete-la ye (Córdoba)</td>
<td>Alvarado - venadillo (Tolima)</td>
<td>El vino - el rosal (Cundinamarca)</td>
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<td>Sibate - fusagasuga (Cundinamarca)</td>
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<td>Mediacaanoa-la virginia (Valle del Cauca)</td>
<td>La calera-la cabaña (Cundinamarca)</td>
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<td>Chiquinquira-tinjaca (Boyacá)</td>
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<tr>
<td>Cerete-lorica (Córdoba)</td>
<td>Toro-el amparo (Valle del Cauca)</td>
<td>Las juntas-santa maria (Boyacá)</td>
<td></td>
</tr>
<tr>
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<td>Chaparral-ortega (Tolima)</td>
<td>Othane-chiquinquirá (Boyacá)</td>
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<td>El banco-San José (Magdalena)</td>
<td>Necoclí-arboletes (Antioquia)</td>
<td>Te de Viani-guayabal (Cundinamarca)</td>
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<tr>
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<td>Chigorodo-dabeiba (Tolima)</td>
<td>Bogotá-choachi (Cundinamarca)</td>
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<tr>
<td>Santa lucia-moñitos (Córdoba)</td>
<td>Bolomombo-santa fe de Antioquia (Tolima)</td>
<td>Tres puertas-ira (Caldas)</td>
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<td>Pacho - la palma (Cundinamarca)</td>
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<td>La frontera-la union (Antioquia)</td>
<td>Garagoa - chinavita (Boyacá)</td>
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<tr>
<td>T de santa rosa-villanueva (Bolívar)</td>
<td>Te argelia-el amparo (Valle del Cauca)</td>
<td>Ramiriqui - miraflores (Boyacá)</td>
<td></td>
</tr>
</tbody>
</table>

### 6 DEVELOPMENT OF TRANSITION PROBABILITY MATRICES

A Matlab® routine was developed to generate the TPMs for this study. The routine followed these steps:

1. Assign a numeric value to the condition of the road section, as follows: Very good (a value of 5), Good (4), Fair (3), Bad (2) and, (1) Very Bad.
Two further conditions were subsequently verified: (1) the road section had data for at least nine consecutive analysis periods (the analysis period was defined as 6 months), meaning that there was no missing data from the INVIA database and; (2) the condition assigned to the road section had to be equal or of decreasing nature with respect to immediate prior period. The road sections complying with these two conditions were included and a label was assigned to these to identify their temperature and traffic categories.

Table 3 presents the TPMs that correspond to the different temperature and traffic scenarios that can be used to characterize the Colombian-road network. Data presented on Table 3 suggests that as the temperature and the traffic increase, the rate at which a pavement sections deteriorates increases. However, it is difficult to compare one TPM to another.

### Table 3 TPM for the different Temperature and traffic scenarios

<table>
<thead>
<tr>
<th>Temperature</th>
<th>Low</th>
<th>Medium</th>
<th>High</th>
</tr>
</thead>
<tbody>
<tr>
<td>9°C - 23°C</td>
<td>0.2254, 0.1662</td>
<td>0.2394, 0.1332</td>
<td>0.0351</td>
</tr>
<tr>
<td>23°C - 30°C</td>
<td>0.0949, 0.1975</td>
<td>0.1720, 0.1592</td>
<td>0.0761</td>
</tr>
<tr>
<td>30°C - 37°C</td>
<td>0.2712, 0.4208</td>
<td>0.1978, 0.0195</td>
<td>0.0889</td>
</tr>
</tbody>
</table>

Criticality index (CI) of a TPM

“Criticality” is defined as the speed at which the condition predicted using a TPM decreases in a given road section. This index is calculated using the Criticality Index (CI) that is referred to an analysis of third order dynamic systems. This methodology could be applicable to several physical phenomena, including pavement deterioration, depending on its compliance with the following constraints: (i) the systems’ physical conditions are uniform and homogeneous; (ii) changes within the system are expressed in a gradual and continuous manner and; (iii) when the final stage/condition is reached, the system stabilizes and no more changes occur. The methodology includes the evaluation of several parameters that include, but are not restricted to: Dead time ($t_0$), which represents the time required for the system to capture changes in the response variable as a result of modifications in the input variables; Gain (k), which indicates the total variation in the condition of the road from its initial condition to the terminal stage; and , $\tau$, is a key factor associated with time required by a process to reach a certain change with respect to its total change (k); as such, this time constant is tied to the process’ speed response. It is important to note that for first-order dynamic systems, $\tau$ indicates the time required for the output variable to reach 63.2% of the total change. In higher-order systems, the time constant’s meaning is not exactly the same. In fact, there are as many $\tau$ parameters as the order of the modeling system and one cannot state that any of the $\tau$ values represents the time to reach 63.2% of the total system change; however, values of $\tau$ in the transfer function are still indicators of the system dynamics. In this study, an effective-time constant ($\tau_{eff}$) was determined by taking the highest value among the other time constants. The following equations correspond to the transfer functions from the dynamic models of: first, second and third order.

$$G(S) = \frac{K e^{-t_0 S}}{(1 + t_0 S)}$$  

$$G(S) = \frac{K e^{-t_0 S}}{(1 + \tau S)(1 + t_0 S)}$$
Several attempts were conducted to characterise the CI of the systems that included first-order, second-order and third-order adjustments as shown in Figure 1. One can see that the best fit is provided by the third-order approach. Furthermore, Table 4 presents the $\tau_{efc}$ parameters obtained from a third order calibration for all the scenarios. The values presented on Table 5 can be interpreted as follows: the CI indicates the “time” needed for a TPM to reach a critical condition, meaning that those TPMs with lower CI values would deteriorate faster than those with higher CI values. Table 5 suggests that as temperature and traffic increase, the CI decreases, this is consistent with the performance exhibited by the pavement sections.

Figure 1 (a) First-order, (b) second-order and, (c) third-order fitting for [A, A] scenario after modeling in Matlab®.

Table 5 Criticality Index (CI) associated to the nine scenarios of traffic and temperature.

<table>
<thead>
<tr>
<th>CI</th>
<th>Temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td>$CI_{[A,A]}$</td>
<td>$CI_{[M,A]}$</td>
</tr>
<tr>
<td>$134.33$</td>
<td>$170.02$</td>
</tr>
<tr>
<td>$IC_{[A,M]}$</td>
<td>$IC_{[M,M]}$</td>
</tr>
<tr>
<td>$274.21$</td>
<td>$274.21$</td>
</tr>
<tr>
<td>$IC_{[A,B]}$</td>
<td>$IC_{[M,B]}$</td>
</tr>
<tr>
<td>$317.08$</td>
<td>$345.96$</td>
</tr>
</tbody>
</table>

7 CONCLUSIVE REMARKS

A detailed analysis of the influence of temperature and traffic on the deterioration of pavement structures was conducted. Transition probability matrices (TPMs) were obtained following the Markov theories to generate characteristic matrices for the entire temperature and traffic spectrum available in the data set of the Colombian road network. The data suggest that the loss of pavement condition occurs more rapidly in scenarios with higher traffic and temperature. Considering the challenge found when attempting to compare the behavior of different TPM associated with a particular scenario, a methodology was proposed to define an index that would define the criticality of a TPM, hence facilitate the process of comparison. A Criticality Index (CI) was proposed, which was based on a third-level dynamic analysis to capture pavement deterioration. IC results were consistent with the trends and deterioration speeds expected for pavement damage evolution.
The developed TPM could be used at the network level by the Colombia Road Institute – (INVIAS) and other road authorities for improving the pavement management systems currently available and incorporating updated methodologies to characterize deterioration and predict future conditions for the road network. Similarly, the decision-making process for budget allocation could take advantage of this tool to make efficient use of available resources and make adequate decisions for maintenance and rehabilitation of the road network, taking into consideration climatic and traffic data.

REFERENCES


The Importance of Developing Maintenance and Rehabilitation Plans for Young Transportation Networks

Development of asset management plans has become a standard part of the road owner process around the world. Many owners of young networks (where bulk of the roads are less than 20 years old) however fall behind in the process, when they are the groups that will benefit the most. Most agencies are not prepared for the shift from new construction to maintenance and end up with a large backlog of expensive rehabilitation projects.

New roads are constructed on freshly compacted/consolidated soils and often use newer materials and technologies that haven’t been seen through to their full life-cycle. By tracking performance, materials, technologies, construction quality indicators, and design differences, each item can be evaluated to determine with statistical significance what are the successes and areas for improvement.

As been the experience with public private partnerships, creating maintenance and operations plans at the outset of a project will ensure funding is available to maintain the deserved level of service over the complete life of the road network. Having the complete level of detail and a plan to monitor what happens to the road will allow you to predict user satisfaction, maintenance plans, and long term budgets with a clear level of accuracy.
1 INTRODUCTION

Transportation asset management is a concept that has been around for many decades now. It has become very common in countries with older, established infrastructure. These countries have shown that the benefits of the asset management system will allow them to do advanced planning over the large network, plan for future maintenance activities, provide justification and clarity to budget requests, and predict realistic network level conditions to achieve the desired level of service.

Many government agencies that have perceived newer or smaller networks are often reluctant to perform the necessary work to create a transportation asset management system as they can see an immediate cost and no immediate benefits. However, many of these agencies are the ones that in fact need an asset management plan the most and will be able to most effectively implement and maintain a plan. These agencies typically do not have clear defined maintenance budgets and instead are very reactive to issues that have been identified by politicians, the public, or the media.

The reactive maintenance systems, the costs of the individual repair often seem reasonable as the right decisions based on current conditions are often made. But the lack of preventative treatment on the silent problems cause the accumulation of a infrastructure debt that can go un-noticed for more than a decade. This has led advanced nations like the United States to receive a grade on their vital infrastructure of a ‘D’ (ASCE, 2017). With spending of over $160 billion USD per year on infrastructure repairs and the conditions show signs of further deterioration. There is a current backlog of needs worth $836 billion USD due to the lack of maintenance and age of the network. This has resulted in a range of new processes that are being mandated by government agencies to better track the spending of public monies and to ensure the focus is on the entire network, not just local repairs.

The process to develop an inventory and monitor usability of transportation networks is not new on the global scale. It is just important to start the process before you reach a deteriorated state to save large future repair costs. The discussion below will outline the steps for a new agency to implement an asset management plan and describe the benefits and how to ensure that the benefits will be reached.

2 DEVELOPMENT OF A TRANSPORTATION ASSET MANAGEMENT PLAN

Development of a Transportation Asset Management Plan (TAMP) is not a single project that can be completed and filed away. It is an on-going process that becomes a living document as the technology, budget, and expectations of the travelling public change over time. This will help ensure that the plan represents the values of the organization and that it is sustainable for the long term.

The Core Principles of Asset Management are (FHWA, 2007):

- **Policy-driven** – Resource allocation decisions are based on well-defined set of policy goals and objectives.
- **Performance-based** – Policy objectives are translated into system performance measures that are used for both day-to-day and strategic management.
- **Analysis of Options and Tradeoffs** – Decision on how to allocate funds within and across different types of investments (e.g. preventive maintenance versus rehabilitation, pavements versus bridges) are based on an analysis of how different allocations will impact achievement of relevant policy objectives.
Decisions based on Quality Information – The merits of different options with respect to an agency’s policy goals are evaluated using credible and current data.

Monitoring Provides Clear Accountability and Feedback – Performance results are monitored and reported for both impacts and effectiveness.

These principles are established to define the elements that make the plan effective and sustainable. It needs to be a document that cleanly defines the methods and indicators that define success for the agency. It provides a level of transparency for tax payers to show areas of success and areas for improvement. This demonstrates to the public, taxpayers, and government officials how public monies are being spent and what can be done to spend them more effectively.

Development of these plans requires a lot of information and discussion in order to get all stakeholders to agree on how to achieve the agency goals. This involves a lot of planning to achieve due to the large number of people involved and the constantly changing conditions. Figure 1 shows a high level diagram describing the inputs and process to create and maintain the TAMP.

![Figure 1. Transportation Asset Management Process](image-url)

3 ADVANTAGES OF A NEW NETWORK FOR ESTABLISHING A TRANSPORTATION ASSET MANAGEMENT PLAN

Transportation networks are very complicated and the maintenance of them is an additional level of complication. Older networks have an evolution of technology and materials along with a much more varied level of condition across the network. This is viewed in the field as a large range in road sections, conditions, materials, repairs, and less documentation.

New networks have several key benefits that can be used when creating an asset management plan. The most obvious benefit is being initially in good condition. This allows for the policy to recommend to maintain the condition and average network condition, which is much more cost effective than trying to improve the conditions.

3.1 Setting a Benchmark for New Construction

Technology and research has pushed the construction of road networks forward dramatically. Many of the improvements have been done to achieve several major goals:

- Reduce the cost of construction,
- Improve the quality at the time of construction,
- Better understand the design life,
- Reduce the number of premature failures,
- Identify the early signs of deterioration for correction.
This has led to changes in design methodologies, material selection, construction equipment, construction technique, quality control procedures, and payment terms for construction. In short, new pavements are much more likely to provide longer terms of service and value to the road owners.

It becomes important early on to set good procurement policies to ensure that the technologies are put to use when effective and that the benefits are realized. This allows for the use of innovation and technology in the design and construction of roads. This must be done in a controlled fashion to ensure that the projects achieve success. Examples of innovation in the construction of new roads includes:

- Use of different procurement practices, such as design/build contracts to encourage innovation,
- Advanced materials (ie. SuperPave, higher strength concrete, polymer reinforcements),
- Advance construction equipment (ie. Material transfer vehicles, roller compacted concrete, smart compaction),
- Quality control (ie. Material sampling, source testing, accelerated material testing, non-destructive testing),
- New design practices (ie. Mechanistic/empirical pavement design, SuperPave mix design, advanced material factors, drainage improvements).

The best thing an agency can do is keep an open mind to new technologies and to determine what might have the largest improvement on the construction of new roads. Policies and acceptance criteria need to be established by the road agencies to allow these improvements, while ensuring they improve the quality and cost effectiveness of the road network.

3.2 Identifying Trends in Premature Failures

One of the most important and difficult parts of implementing a TAMP is the need for data driven decisions. In many cases there is a lot of valuable information, but it is not properly organized in order to make decisions that can have a significant impact on the network. Poor quality sections are often not repaired properly and as such become the most costly repairs as they are completed over and over again with maintenance.

One of the most common examples of this is the appearance of premature failures on newly constructed infrastructure. Most bridges have design lives in excess of 50 years and pavements have typical initial lives greater than 20 years. However, localized problems commonly occur within the first 5 years for these and other types of infrastructure. While some deterioration is expected early on (ie. extreme climate and traffic are anticipated to cause early changes), that is different than a failure which was not anticipated.

Typical reactions for these kinds of problems involve the immediate repair or replacement of the problem without an investigation. The cause of the premature failures are linked to a small list of potential causes including:

- Poor construction practices,
- Variations in construction materials from the design,
- Inadequate site investigation / design not matching conditions.

If the root cause of the problems are investigated for many of these issues, they can be prevented. Common adjustments to the existing specifications include (but is not limited to):

- Improvements to the acceptance testing,
- Changes to material specifications,
- Increased monitoring during construction,
- Changes in recommended practices,
- Changes to an international design process to account for local materials and or conditions,
- Adequate incentives for quality construction practices.

The important things here is to keep a clear data repository that is easily referenced and searched to describe problems encountered. Many trends are easily identified with specific materials, aggregate sources, construction companies, seasonal factors, or design methods. These can lead to simple and effective improvements that can prevent problems with long term and expensive repercussions. This can lead to changes of policy and adjust procurement and quality processes to prevent some of the common sources of failure and greatly increase the quality of the network for minimum costs.

3.3 Establishing Performance Models and Comparison with Design

Most larger forms of infrastructure have a long term design and an expected path of deterioration. Modern design take into account many aspects of risk including a probability of failure, expected life, and a design life. These
factors are often described in great details during the planning phase, but are not monitored after construction. Having a clear process to monitor this will provide much insight into the design process and any issues that are not well addressed by the process.

This is often represented by a design life and can be modelled using a range of techniques. Techniques for modelling performance range from simple time based deterministic deterioration (such as straight line or sigmodal deterioration) through to more complicated probabilistic methods that work well for items with sudden changes in condition. These methodologies will all work to demonstrate what existing highway sections will look like over time. This process is described in more details below with the Life Cycle Cost Analysis.

To track the real world performance, it is important to have real time information about current network conditions. You cannot fix problems unless you know they exist, so this is one of the most important parts of the process. There are a range of techniques for this level of monitoring and many global standards for the evaluation of pavements and bridges, which represent most of the value in the infrastructure being reviewed. Figure 2 shows example tools for how the monitoring of existing conditions can be performed.

The inspection of the infrastructure is to complete several important goals:

- Maintain a complete inventory,
- Track overall network condition for performance goals,
- Identify immediate safety concerns,
- Predict future conditions for larger maintenance activities,
- Detect sections where preventative maintenance can be applied cost effectively.

There are numerous manufacturers of this equipment and service providers that can ensure you have a custom plan to meet the needs of your roads. This includes measuring specific signs of distress that are relevant to your network only. A clear plan to ensure correct and consistent data will ensure you know the true condition of your network and where you are compared to your goals.

4 PERFORMANCE BASED DECISIONS AND THEIR INFLUENCE ON NETWORK CONDITION

To actively manage a large network of assets, it is important to have up-to-date information on the assets. This allows for timely decisions to be made that keeps all assets running in the desired state. If data is not available on many of the assets, it is possible for them to be deteriorated to the point of failure without the owner’s knowledge. This often leads to very expensive repairs and angry end users that are no longer able to get where they want to go in a safe and timely manner.

4.1 Preventative Maintenance and the Need for Early Detection

Preventative maintenance is a very simple concept which applies to many areas of life, including transportation infrastructure. Many problems have early warning signs that a larger problem is about to occur. By fixing the problems at this stage, the costs can be greatly reduced. Examples of this can be seen in many aspects of transportation
such as oil changes to prevent engines from seizing, crack sealing to prevent material loss, and painting of steel bridges to prevent corrosion.

The key to an effective preventative maintenance program is the early detection of issues and the ability to react in a timely manner to prevent the rapid deterioration that is expected to occur soon. This is often as simple as identifying deteriorated paint or small cracks that are still readily visible. The problem becomes recognising all of these signs on networks with over 10,000km of highway and over 1,000 structures.

It is vitally important to identify these issues as early as possible. For example, the cost to complete an oil change on an engine is relatively minor if it is done while the engine is in good condition. But if neglected, the oil level could drop or the viscosity could be reduced causing unnecessary friction. This leads to the potential of needing to replace large portions of the engine with a cost over 100 times higher. Not only is the cost higher, but there is the inconvenience as the engine has a catastrophic failure when it is in use, causing long delays during repairs and delays to all of those that depend on the vehicle. These types of issues are readily identified for almost all types of transportation infrastructure.

4.2 Life Cycle Costing

The benefits of preventative maintenance are easy to quantify and this has led to the success of many uses of life cycle cost analysis. This is a common practice which tracks the anticipated flow of expenditures and conditions over time to quantify the overall impact of making decisions with complex long term impacts.

The process to complete life cycle cost analysis is well documented (FHWA, 2008) and can be done quickly with a little bit of experience. The goal is to account for all costs that are anticipated over the lifetime, or portion of the lifetime, of the asset. This involves determining the initial cost of construction, all relevant maintenance and rehabilitation work, other incidental costs that may vary over time, and often times a salvage value at the end of the time period, just to ensure a fair comparison. If a comparison is being made between several alternatives, it is customary to leave out items that will not vary between the options.

To complete the analysis, it is important to get a clear cost and timings of all of the relevant activities. This is commonly done diagrammatically to show the magnitude and timing of the various expenses. As can be seen in Figure 3, the initial costs are most often the largest individual cost. Major maintenance usually represent the next most significant costs in the process. Routine maintenance are often small individually, however are expected to occur frequently. It is also possible to add additional properties and costs. Figure 3 also shows that user costs can be included, which may vary (ie. increase) based on the level of road deterioration.

![Figure 3. Life Cycle Cost Diagram](image-url)
broken down as shown in Table 1. This is an example in Canadian Dollars, to estimate the original construction costs of a 2 lane rural arterial highway (ARA, 2008).

Table 1. Sample Estimate for the Construction of a New Pavement

<table>
<thead>
<tr>
<th>Pavement Layer</th>
<th>Description of pavement layer, Amount (Quantity)</th>
<th>Amount</th>
<th>Quantity</th>
<th>Price per unit of quantity</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface</td>
<td>Superpave 12.5 mm FC1, mm (t)</td>
<td>40</td>
<td>750</td>
<td>$6.75</td>
<td>$51,562.50</td>
</tr>
<tr>
<td>Binder</td>
<td>Superpave 19 mm, mm (t)</td>
<td>60</td>
<td>1125</td>
<td>$53.70</td>
<td>$60,412.50</td>
</tr>
<tr>
<td>Extra Layer</td>
<td>Superpave 19 mm, mm (t)</td>
<td>60</td>
<td>1125</td>
<td>$53.70</td>
<td>$60,412.50</td>
</tr>
<tr>
<td>Base</td>
<td>Granular A, mm (t)</td>
<td>150</td>
<td>6991</td>
<td>$13.01</td>
<td>$90,952.91</td>
</tr>
<tr>
<td>Subbase</td>
<td>Granular B, mm (t)</td>
<td>450</td>
<td>15206</td>
<td>$6.71</td>
<td>$102,032.26</td>
</tr>
<tr>
<td>Shoulder Surface</td>
<td>Superpave 12.5 mm FC1, mm (t)</td>
<td>40</td>
<td>100</td>
<td>$68.75</td>
<td>$6,875.00</td>
</tr>
<tr>
<td>Shoulder Base</td>
<td>Superpave 19 mm, mm (t)</td>
<td>50</td>
<td>125</td>
<td>$53.70</td>
<td>$6,712.50</td>
</tr>
<tr>
<td>Subgrade</td>
<td>Subgrade Improvement, % area (m²)</td>
<td>500</td>
<td>$5.00</td>
<td>$2,500.00</td>
<td></td>
</tr>
<tr>
<td>Tack Coating</td>
<td>Tack coating, number of applications (m²)</td>
<td>1</td>
<td>7500</td>
<td>$0.26</td>
<td>$1,950.00</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$383,410.17</strong></td>
</tr>
</tbody>
</table>

The value for the various options are often adjusted for inflation and interest over time as well. This allows for a fair comparison of costs over time. So this is usually accomplished by assigning activities to a year of implementation into the future. The quantity and unit costs can then be adjusted with a discount rate to determine what the equivalent present value of all of the rehabilitation is. As can be seen in Table 2, the future costs can be discounted to account for the planned time in the future to prevent a net present worth. This will allow for a fair comparison between options that have major expenses that are further out in time.

Table 2. Sample Estimate of the Life Cycle Costs for Future Maintenance

<table>
<thead>
<tr>
<th>Years after initial construction</th>
<th>Description of pavement layer, Amount (Quantity)</th>
<th>Amount</th>
<th>Quantity</th>
<th>Price per unit of quantity</th>
<th>Cost</th>
<th>Net present worth</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Rout and seal, m (m)</td>
<td>500</td>
<td>$2.00</td>
<td>$1,000.00</td>
<td>$747.26</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Spot repairs - mill and patch, % Area (m²)</td>
<td>10</td>
<td>750</td>
<td>$7.70</td>
<td>$5,775.00</td>
<td>$3,418.21</td>
</tr>
<tr>
<td>15</td>
<td>Mill AC, mm (t)</td>
<td>40</td>
<td>750</td>
<td>$11.00</td>
<td>$8,250.00</td>
<td>$3,442.44</td>
</tr>
<tr>
<td>15</td>
<td>Resurface with SP 12.5mm FC1, mm (t)</td>
<td>40</td>
<td>750</td>
<td>$68.75</td>
<td>$51,562.50</td>
<td>$21,515.23</td>
</tr>
<tr>
<td>15</td>
<td>Resurface with SP 19mm, mm (t)</td>
<td>50</td>
<td>938</td>
<td>$53.70</td>
<td>$50,370.60</td>
<td>$21,017.89</td>
</tr>
<tr>
<td>19</td>
<td>Rout and seal, m (m)</td>
<td>500</td>
<td>$2.00</td>
<td>$1,000.00</td>
<td>$330.51</td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>Spot repairs - mill and patch, % Area (m²)</td>
<td>20</td>
<td>1500</td>
<td>$7.70</td>
<td>$11,550.00</td>
<td>$3,817.43</td>
</tr>
<tr>
<td>27</td>
<td>Mill AC, mm (t)</td>
<td>40</td>
<td>750</td>
<td>$11.00</td>
<td>$8,250.00</td>
<td>$1,710.79</td>
</tr>
<tr>
<td>27</td>
<td>Resurface with SP 12.5mm FC1, mm (t)</td>
<td>40</td>
<td>750</td>
<td>$68.75</td>
<td>$51,562.50</td>
<td>$10,692.41</td>
</tr>
<tr>
<td>27</td>
<td>Resurface with SP 19mm, mm (t)</td>
<td>50</td>
<td>938</td>
<td>$53.70</td>
<td>$50,370.60</td>
<td>$10,445.25</td>
</tr>
<tr>
<td>31</td>
<td>Rout and seal, m (m)</td>
<td>500</td>
<td>$2.00</td>
<td>$1,000.00</td>
<td>$164.25</td>
<td></td>
</tr>
<tr>
<td>35</td>
<td>Spot repairs - mill and patch, % Area (m²)</td>
<td>20</td>
<td>1500</td>
<td>$7.70</td>
<td>$11,550.00</td>
<td>$1,502.12</td>
</tr>
<tr>
<td>38</td>
<td>Mill AC, mm (t)</td>
<td>40</td>
<td>750</td>
<td>$11.00</td>
<td>$8,250.00</td>
<td>$901.22</td>
</tr>
<tr>
<td>38</td>
<td>Resurface with SP 12.5mm FC1, mm (t)</td>
<td>40</td>
<td>750</td>
<td>$68.75</td>
<td>$51,562.50</td>
<td>$5,632.63</td>
</tr>
<tr>
<td>38</td>
<td>Resurface with SP 19mm, mm (t)</td>
<td>50</td>
<td>938</td>
<td>$53.70</td>
<td>$50,370.60</td>
<td>$5,502.43</td>
</tr>
<tr>
<td>42</td>
<td>Rout and seal, m (m)</td>
<td>500</td>
<td>$2.00</td>
<td>$1,000.00</td>
<td>$86.53</td>
<td></td>
</tr>
<tr>
<td>42</td>
<td>Spot repairs - mill and patch, % Area (m²)</td>
<td>20</td>
<td>1500</td>
<td>$7.70</td>
<td>$11,550.00</td>
<td>$999.39</td>
</tr>
<tr>
<td>48</td>
<td>Mill AC, mm (t)</td>
<td>100</td>
<td>1875</td>
<td>$11.00</td>
<td>$20,625.00</td>
<td>$1,258.09</td>
</tr>
<tr>
<td>48</td>
<td>Resurface with SP 12.5mm FC1, mm (t)</td>
<td>40</td>
<td>750</td>
<td>$68.75</td>
<td>$51,562.50</td>
<td>$3,145.23</td>
</tr>
<tr>
<td>48</td>
<td>Resurface with SP 19mm, mm (t)</td>
<td>60</td>
<td>1125</td>
<td>$53.70</td>
<td>$60,412.50</td>
<td>$3,685.07</td>
</tr>
<tr>
<td>53</td>
<td>Rout and seal, m (m)</td>
<td>500</td>
<td>$2.00</td>
<td>$1,000.00</td>
<td>$45.58</td>
<td></td>
</tr>
<tr>
<td>57</td>
<td>Spot repairs - mill and patch, % Area (m²)</td>
<td>10</td>
<td>750</td>
<td>$7.70</td>
<td>$5,775.00</td>
<td>$208.51</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>$514,349.30</strong></td>
<td><strong>$100,269.07</strong></td>
</tr>
</tbody>
</table>
4.3 Alternative Comparisons

One of the biggest advantages of the LCCA process is that you can evaluate the potential impact of competing alternatives, designs, materials, and maintenance strategies.

A common practice for high traffic areas is to consider the benefit of an asphalt pavement as compared to a concrete pavement. In this case, the initial construction costs can be very different depending on the design. But often times the activity with the lower initial cost will require more funding later to maintain the conditions at a high level.

In some cases, agencies will complete what is known as an alternative bid project. In this case, an LCCA for both an asphalt and a concrete design would be created. Since the LCCA is created based on assumed pricing for many of the items, some variation is anticipated as compared to the unit costs submitted as a part of a bid. In this case, the initial construction costs are often used to replace the assumed initial cost, but the contract award is actually based on the total LCCA and not just the initial construction.

In addition to the cost comparison, conditions can also be compared. As can be seen below in Figure 4, the condition of the two alternatives is expected to have a very different path. Assuming that Alternative A and Alternative B both have the same cost, the average performance is not the same. So agencies may prefer to implement Alternative A which has a larger area beneath the deterioration curves, representing a higher level of performance.

![Figure 4. Comparison of Performance between Alternatives.](image)

This type of comparison works very well when doing an initial design of a pavement or completing a value engineering phase. By comparing the simulated costs over time, the right alternative for each agency’s policies for capital costs, maintenance costs, user performance, and financial planning can be addressed.

4.4 Network Level Life Cycle Costing

This type of planning can be done section by section and when properly implemented, be performed network wide. With a clean inventory and existing performance information, these types of data driven decisions can be made to decide where funds should be spent and how much budget needs to be allocated for future years.

There are a range of network level asset management tools that are commercially available that can be used to predict the future costs, as well as future performance. These systems are available for a range of individual assets and will work to do a range of what-if scenarios. For example, there are numerous pavement management software tools that will allow agencies to perform the following types of analysis:

- Determine the long term cost to maintain existing conditions,
- Determine costs to achieve a certain performance condition,
- Determine the performance impact of maintaining or altering a budget over time,
- Estimate of the needed budget to eliminate all major defects and/or safety issues.

There is a large range in the capabilities in these systems and to what level they are able optimize the spending and the network performance. Comparing the potential of applying any feasible project in one year has a finite number
of solutions, but comparison between years, and between asset types has created some substantial computational challenges.

The advantage of these scenarios can be readily viewed using these tools as well. By adjusting the assumptions in the prediction, a worst-first set of priorities can be simulated (Menendez & Narciso, 2012). This will show the difference in cost if the preventative maintenance is not completed and agencies wait till there are significant problems before addressing.

5 PLANNING FOR AN OLDER NETWORK

Many agencies are able to budget for new development and capital projects as the costs are clearly defined. But when building highway and road networks, the infrastructure is expected to stand the test of time and last for an indefinitely into the future. As such is important to plan not just for the initial construction, but to set aside on-going funds for the performance of preventative maintenance and future rehabilitations. This has been an issue that is not often identified early and budgets are not in place when necessary. This results in having to perform an expensive treatments rather than the timely implementation of maintenance.

5.1 Prediction of Network Needs

The life cycle cost analysis works very well for single construction projects. While it is possible to complete this level of effort for entire networks, but often times the network is in a range of conditions from brand new roads through to roads requiring advanced repair. As such there are a few ways to estimate the needs of the entire network.

Pavement management systems have been designed specifically for this purpose. These software tools are capable of automating the creation of LCCA for each road segment in large networks. They can also help by using other information such as current condition to improve these estimates for the network and provide robust budgets and conditions for networks of all sizes. These systems greatly range in terms of the level of detail, ability for future prediction, and analysis options.

5.2 Prediction of Budget to Meet Desired Level of Service

Most agencies have a pretty good feeling of the level of the maintenance that they are currently providing for their network along with an idea if it is enough to maintain their network. Many agencies find themselves spending their complete budget, but with the clear impression that the network is deteriorating faster than it can be maintained. But most agencies don’t have a clear understanding of what their budget should be.

There are several ways that a budget can be estimated. One of the fastest, but relatively high level methods is to use the life cycle cost analysis for a small sampling of the entire network. For example, if the LCCA has been created for an arterial road, it will be able to use this to estimate the average annual cost after construction. But average the cost in all future years (including the salvage), a typical cost for a 1km sample of road can be determined. Assuming that the network is equally represented by the range of conditions, the budget can be estimated to be the product of the average annual cost and the road network length. Even for newer networks, this type of budget can likely represent a steady state budget to represent the conditions in the long term of the network. Many times giving a target budget will allow the agencies to work towards achieving the correct level of funding.

Pavement management systems are also designed specifically to tackle the network level implementation of the LCCA. By looking at the existing conditions, and predicting future conditions, a range of treatments can be selected as feasible and costs assumed.

6 CONCLUSIONS

The purpose of a Transportation Asset Management Plan is to ensure that the resources are available when required to meet the vision and the goals of a road or a highway network. These plans are living documents that are time consuming to create, but pay back benefits that greatly outweigh the effort. Many established agencies in North America and Europe for implemented these systems and are great examples to follow. They have encountered and found solutions for the many challenges in creating policy that aligns with the vision, goals, resources, and assets of an agency.

By establishing an initial set of goals and understanding current conditions there is a lot that can be done. Many agencies with rapidly growing networks with younger ages often neglect planning for the future. These agencies need to understand exactly what is the planned maintenance and rehabilitation budget for their network in 20 to 50
years. Is there a specific plan to ensure finding is available at that time? Does the plan include active maintenance of the network to ensure cost effective measures that maximize the conditions for the end users?

With a range of available industry knowledge and tools, young agencies are better equipped to address this need now. This will enable them to make performance based decisions that are driven by policy. With regular updates to information on the current condition of their assets, agencies can review all options and trade-offs to ensure that the funding is being spent on their priorities. The transparent monitoring of this plan will also allow agencies to get the timely feedback they need and to ensure that the goals are being met and the infrastructure enables the economy to grow.

7 REFERENCES


The impact of overloaded Trucks on road infrastructures and needs to reactivation of the 1986 law (Heavy Vehicles Axle Load) in Abu Dhabi Emirate.

**ABSTRACT:**

An overload, in commuting system, is defined as a load that exceeds the legal truckload limit. The overload statistics depend on truck types specified by the number of axles. The truckload spectra generally represent a distinctly different pattern than that in lower load levels, which is especially true among trucks with five or more axles. The probability of occurrence of specific truck weights exceeding the legal load limit can be used to estimate the frequency of occurrence of heavier loads in transportation facilities and thus determine the damage potentials of overloads on the infrastructure. This information can be used along with routine engineering analyses that many pertinent road agencies conduct when issuing truck-overload permits.

The Heavy Vehicle Axle load regulations known as UAE “1986 Law”, has not been put in enforcement since the time of its inception. However, the current situation with regard to truck weights in the UAE has negative implications for road safety and puts excessive stress on road infrastructure. This leads to more road traffic accidents, higher maintenance expenditure and risk of structural failures.

This research will conduct a comprehensive study to evaluate the implications on the network performance including the structural capacity to control the impact from heavy commercial vehicles. The investigation will evaluate the following:

- Design specification of the current road network;
- Specifications of the trucks;
- Road maintenance implications;
- Road safety & industry costs implications;
- Traffic volumes on the road network.

This paper aims to exchange knowledge and share information of the implication of overloaded trucks not only in UAE but also in GCC region and worldwide.
The impact of overloaded trucks on road infrastructure and needs to reactivation of the 1986 law (Heavy Vehicles Axle Load) in Abu Dhabi Emirate

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1. INTRODUCTION

Overloaded heavy commercial vehicles (OHV) is a worldwide issue that affects the infrastructure massively. OHV creates serious threats to the transport infrastructure, such as: a) Accident risk and accident severity (Truck instability, braking default, tire overheat and loss of manoeuvrability) b) Damage to the infrastructure (Permeant pavement failure) c) Economic impact (Large distortions in freight transport competition). (Jacob & Feypell-de La Beaumelle, 2010).

Heavy vehicles and road safety data has been extensively investigated in Australia, Europe and the Americas, but not much information is available to date for the United Arab Emirates (UAE). Heavy vehicle usage on local roads is a key factor contributing to national growth for UAE’s economic development, which makes OHV operations and management an important issue within the field of transportation. Abu Dhabi emirate have recently introduced programs to monitor axle load limits. However, heavy vehicles operations are not strictly regulated in the emirate and the most appropriate regulatory limits to be enforced are still unused. Also, there is lack of formal data and research on OHV effect on the infrastructure issue, which is considered a foundation for the emirate to build new regulations, policies and strategies for OHV.

The purpose of this paper is to demonstrate the need to investigate the 1986 law for trucks weight limits that shape the heavy vehicles safety framework by determining the probability of occurrence of specific truck weights exceeding the legal load limit to estimate the frequency of occurrence of heavier loads in highways and thus determine the damage possibilities of overloads to the infrastructure and propose new amendments to the 1986 law based on the outcomes of this study. As the research in it is earlier stages, two proposed methodologies might be applied to achieve the objectives of the study:

- Statistical evaluation of truck overloads
- Site survey and analysis of highway trucks overloading status quo in Abu Dhabi city

Based on the outcomes of the research one of the following three strategies might be an option for the city to overcome the issue:

a) Adopting an innovative approach of Performance Based standards (PBS) of heavy vehicles by reflecting it on the 1986 law
b) Improving the infrastructure quality itself by enhancing the pavement requirements.
c) Applying conformity schemes programs for heavy vehicles drivers to improve their truck behavior.

Further strategies, amendments and details will be discussed and introduced during research progress.

2. LITERATURE REVIEW

2.1 Abu Dhabi Emirate and the UAE

The Emirate of Abu Dhabi is the largest in the UAE and comprises approximately 88% of the UAE land area. Series of ambitious and integrated strategies are designed by Abu Dhabi government to achieve a long term competitive advantage at a global level through diversification of the economic plans. One of the main important strategies is developing an efficient transportation infrastructure that meets the sustainable development goals.

Sheikh Zayed Bin Sultan Al Nahyan (Rest upon his soul), the founder of the UAE, established a federal law in 1986 (Heavy Vehicles Axle Load, Law. No 8) to ensure that OHV weights are enforced by the government to prevent any loads exceeding the permitted limits. The law was deactivated since 1990 due to lack of resources and materials required at that time. The current situation of heavy vehicles weights in the UAE has negative impact on the road safety
and puts excessive stress on road infrastructure. This leads to more road traffic accidents, higher maintenance expenditure and risk of structural failures.

In 1994 a heavy vehicle loads and dimensions control system project was initiated in the emirate of Abu Dhabi. The project main aims were to: (a) Control and monitor overloaded heavy commercial vehicles. (b) Establish a database for planning and design use of roads. In 2000, the project was completed and in November 2011 it started to operate officially. (El-qutob & Sharif, 2001). Currently, there are four vehicles monitoring and controlling weigh-in-motion stations in the emirate of Abu Dhabi which are: Mussafah, Khatim-Al Shakla-Al Ain, Saih Ash Sheib and Algweifat border post. The data collected from the stations are very useful in determining the percentage of violations in the emirate. However, new asset management strategies and policies should be implemented to prevent further damage to the infrastructure. According to the Department of Municipal Affairs and Transport (DMAT), Abu Dhabi. The main reasons behind OHV issue are:

a) The suspension of 1986 law since 1990;
b) Lack of effective controls and educational programs for drivers and trucks;
c) Heavy vehicles standards issues (Per ton plus for trucks with two axles and three axles causing more damage than trucks with multiple axles);
d) Limited legal enforcement;
e) Lack of formal and effective data on OHV.

The following statistics shows the current road status in the emirate of Abu Dhabi: a) 60% of truck weights exceed legal limits b) The risk of death in a truck collision is 62% compared to 2.8% in Britain c) A study of 517 random sample highlighted that 100% of drivers and trucks did not pass the traffic safety test although drivers were assured that the last periodic maintenance of trucks did not exceed three months d) 62% of the trucks in the sample were overloaded. Based on these statistics, the emirate of Abu Dhabi is facing the following challenges for OHV issue: a) Lack of sufficient companies with experience in the operation and management of pivotal weight stations b) The urgent need for the comprehensive rehabilitation and maintenance of the central weight station building and upgrading of the weight system c) Functional cadre of judicial control. (Unpublished report, DMAT).

Heavy vehicle usage is important for the national growth and economic development of UAE as transportation is a crucial component which directly accounts for a significant proportion of gross domestic product (GDP) and contribute to almost all other sectors within the country.

The UAE transportation federal management is operated by the Federal Transport Authority-Land and Maritime (FTA). The occurrence of overloaded truck traffic can be an evidence of rapid development of a nation’s economy. The economy of the UAE has grown rapidly and freight demand has increased at the same time. However, the road network cannot bear continuous violations of OHV which is leading to pavement damage and excessive economic costs on the government. Companies and truck drivers have tended to overload their trucks in almost all the UAE (Abu Dhabi, Dubai, Sharjah, Ajman, Umm Al Quwain, Ras Al Khaimah and Fujairah). The percentage of violations increase in the northern emirates due to lack of weigh stations, legal enforcement and monitoring programs.

2.2 Gulf Cooperation Council (GCC) countries

Heavy vehicle usage is an important mode of goods transportation in GCC countries. The huge linked roads between GCC mean that heavy vehicles are more exposed to violating the legal load limits if no effective legal enforcement is applied. With current enforcement laws and fines on heavy vehicles in GCC countries; there are still violations of OHV issue.

There is lack of formal and sufficient updated data for OHV within GCC countries. However, there are some analysis and research that has been conducted addressing the issue in Saudi Arabia and Oman. In Saudi Arabia, optimizing truck weigh stations locations on the highway network in the country was studied and the investigation of the weigh stations showed that truck fleet owners attempt to reduce their unit transportation cost (SR/ton-km) by loading their trucks to maximum capacity. These heavily loaded trucks have resulted in significant road damage. As a result, Saudi Arabia government established a trucks weight monitoring and control program since 1985. (AlGhadhi, 2001)

In Oman, most of the studies were conducted regarding heavy vehicles drivers travel behavior and how their lack of effective education, training and competencies results in continuous violations of OHV on Oman highways. The result of a recent study identified ten themes that were related to common behaviors/risk factors in the heavy vehicle industry in Oman, one of the main important themes was the overloading issue. (AlBulushi, 2017)
The above studies show the need for further investigation for OHV issue and there is still a gap to be studied and filled with accurate data and results to support the problem, as the main gap when it comes to OHV in GCC countries would be the lack of data and statically analysis studies that prove the damage caused by OHV to the infrastructure, the economic cost and the needed strategies, polices and legal frameworks. This study will be a gateway to fill this gap by providing an in-depth literature review about the situation in GCC countries and provide an accurate database about the current OHV situation in the UAE which will lead to a proper legal management enforcement that other GCC countries can use and benefit from to increase the road highway safety and provide a sustainable transport infrastructure.

2.3 Worldwide

Highway agencies all over the world recognize that overweight trucks are a major cause of permeant pavement and bridge deterioration. Developed countries such as Australia and United States of America (US) has extreme high enforcement and inspection programs on heavy vehicles to ensure an effective operation. Several studies have investigated the effects of OHV on the infrastructure as well as Weigh in Motion (WIM) technology enforcement advantages. Two specific have been proposed such as the a) Statistical Analysis using probability distribution models (Mohammadi & Shah, 1993) and b) Numerical Model (Han et al., 2015) c) Site Surveys (Lou, 2005) to analyze the same.

These studies proved that OHV is a serious issue which significantly affects pavement performance and bridge safety. Australia being one of the leading countries in heavy vehicles regulation has adopted the performance based standards (PBS) approach. In the PBS approach, regulators will set out the performance, outcome and level required which gives more flexibility to the designer and the project executor to identify solutions and best practices to select performance requirements for heavy vehicles, such as critical safety areas. PBS approach increase innovation, flexibility and the use of new technology. Unlike the perspective based standards which are rigid specifications for compliance which reduces productivity and has negative implications for road safety. To adopt the PBS approach in Abu Dhabi city, law No 8 of 1986 must be amended to incorporate the new proposed standards and related fines, such as truck weights and dimensions limits, scope of regulation and penalty system. (Adam Ritzinger, Peter Eady, 2010)

3. DATA GATHERING AND PREPARATION

In UAE, limited formal data is available on OHV issue, and is mainly drawn from the Weigh in Motions (WIM) reports. The effect of OHV on Abu Dhabi emirate infrastructure is unclear without further breakdown of the official data, necessary for studying the underlying factors associated with OHV. This will assist in describing the key facilitators and barriers to safe operation and management of OHV issue. Any improvements in OHV safety would be of great national interest and benefit the government and the public. It is anticipated that this research will provide a strong knowledge platform to develop effective interventions and policymaking to reduce OHV effect on the infrastructure.

3.1 Road user’s safety

The UAE government considers accidents and injuries caused by heavy vehicles on UAE’s roads a public serious concern. The risk implications of overloading are not limited to the infrastructure damage only, but it can take human lives and create serious threats to the road safety users. A recent study conducted by DMAT showed that the risk of death in a truck collision is 62% compared to 2.8% in Britain. Accidents and injuries data caused by OHV will be collected through the Accident Investigation System from Abu Dhabi Police to investigate if the OHV is related to high number of death due to truck collision in the UAE.

3.2 Standards and Specifications

Current applied standards and specifications in Abu Dhabi emirate for OHV will be collected from DMAT. The standards will identify the maximum allowable length, width, height, gross weight and axle load for trucks. Additional requirements might be collected from Emirates Standardization and Metrology Authority (ESMA). Possible standards for investigation are:

- DMAT roadway design manuals.

3.3 Geographic Information System (GIS)
GIS information data will be collected from DMAT to investigate and determine the trucks routes in the Emirate of Abu Dhabi. A specific route will be chosen for further investigation based on the frequency, traffic occurrence, pavement damage and rate of accidents of heavy vehicles on that route.

3.4 Weigh in Motion Stations (WIM)

WIM data will be the main source of data for this research. The WIM data will be collected from DMAT for the four active WIM stations in the emirate of Abu Dhabi located in a) Mussafah b) Khatim-Al Shakla-Al Ain c) Saib Ash Sheib d) Algweifat border post. The stored loads, axle spacing, speed, volume, mean value, standard deviation, coefficient of variance and overloading frequencies of vehicles data collected for each truck that passes through the WIM stations will provide a detailed traffic data, such as axle load spectra.

4. METHODOLOGY

To enhance the overloading enforcement 1986 law by evaluating the current state of overloading in the emirate of Abu Dhabi. Also, this study will identify ways in which vehicle inspection systems can be deployed to enhance the highway services for both the user and the government. As the research is in earlier stages there are two different methodologies that might be used to achieve the research goals which are illustrated below:

4.1 Statistical evaluation of truck overloads

The probability of occurrence of specific truck weights exceeding the legal load limit will be used to estimate the frequency of occurrence of heavier loads in highways and thus determine the possibilities of damage to the infrastructure due to overloads. An investigation of truckload data will determine to what extent the overloads are responsible for damage. This analysis will identify the frequency of overloads for certain truckload categories. (Mohammadi & Shah, 1993)

The probability of occurrence of a sequence of events can be determined using Binomial Distribution. For example, the following formula can be used:

\[
P(x"overloading") = \frac{n!}{x!(n-x)!} P^x(1-P)^{(n-x)}
\]

Where three assumptions are required:
- Each replication of the process results in one of two possible outcomes (overloaded or not overloaded),
- The probability of overload is the same for each replication, and
- The replications are independent, meaning here that an overload in one heavy vehicle does not influence the probability of overload in another. (Douglas C. & George C., 2014)

4.2 Site survey and analysis of highway trucks overloading status quo in Abu Dhabi city

A comprehensive survey will be developed to provide a statistical summary of Abu Dhabi highway traffic load data and to sum up the significant characters of overloading transportation system in Abu Dhabi. The site survey will consist of two surveys:

- Vehicle weight survey:
  Different heavy vehicles classes will be included in the vehicle weigh survey. The gross vehicle weight and per axle weight for each truck will be obtained and recorded by the WIM equipment.

- Questionnaire
  Vehicles information, plate number, goods category and other information of weighted trucks will be inquired and collected using a questionnaire. In addition, with the cooperation of DMAT, questionnaire surveys will be assigned to transportation management divisions in Abu Dhabi city to gather the source data of vehicle operating cost and vehicle life. (Lou, 2005)
5. **OUTCOMES**

   It is anticipated that this research will contribute positively not only to the economy of Abu Dhabi but also will have implications to address overloading issue that will impact overloading trucks cost in UAE cost of operation and maintenance of road infrastructure and will enhance the safety of the road users.

   The possible outcomes are as following:
   
   - New strategies, policies and frameworks which will reflect directly on the 1986 law (Heavy Vehicles Axle load, law No.8) that will create a legal management framework for OHV issue.
   - The cost associated with vehicle overloading will be reduced through effective control measures.
   - The re implementation of the new amended 1986 law will lead to operating environments that promote fair competition between service providers and reduced costs of infrastructure maintenance.
   - The proposed approaches will lead to increase in efficiency of transport operations and better facilitation of trade.
   - Policy and recommendation for government and private sectors will be provided.

6. **ACKNOWLEDGEMENT**

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7. **REFERENCES**


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PAPER TITLE: INTEGRATING STRUCTURAL AND FUNCTIONAL EVALUATION OF HIGHWAYS IN OVERALL PAVEMENT MANAGEMENT SYSTEM

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KEYWORDS:
Pavement Management System; Pavement Structural Evaluation; Falling Weight Deflectometer; Remaining Service Life; Maintenance and Rehabilitation Strategies.

ABSTRACT:
The concept of highway asset management (HAM) holds tremendous promise providing a comprehensive business-oriented strategy for managing highway infrastructure. It integrates financing, planning, engineering, personnel, and information management in the decision-making process. As one of the essential component of HAM, pavement management system (PMS) provides objective information and useful data analysis to make consistent, cost-effective and defensible decisions related to the pavement preservation at network and project level. Pavement functional and structural data is required to evaluate pavement through performance monitoring using functional performance indicators including, pavement roughness or ride quality, surface distress, rutting, skid resistance and pavement structural performance indicators includes deflection data. Although pavement structural evaluation (PSE) must be monitored to yield benefits for life-cycle costing at network level PMS but a real anomaly exists most PMSs focus only on functional evaluations of the pavements. This study however, presents network level structural evaluation and its integration into a PMS, and how it can be used as part of a performance prediction tool to allow for more accurate forecasting of the pavement needs over a longer analysis period and help in selecting the appropriate treatment strategy at project level. Structural testing using falling weight deflectometer (FWD) is used to assess current condition as well as to predict future performance of existing pavements and can be an integral part of the decision making process as a component of the PMS. In addition, traffic data (truck load repetitions) will be used to estimate the remaining service life (RSL) and effective structural capacity of the pavement structure. Regular monitoring of structural capacity of pavement is intended to enhance the overall effectiveness of agency’s decisions about maintenance and rehabilitation (M&R) to ensure optimal utilization of available financial resources. It is envisaged that integrating structural evaluation with functional evaluation into overall framework of PMS would provide systematic and objective procedures for monitoring and evaluating pavement performance, selecting optimal type of treatment/ its thickness design, and application time.
Integrating Structural Evaluation of Highways in overall Pavement Management System using Non-Destructive Testing

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I INTRODUCTION

Highway asset management (HAM) covers all important facets by integrating financing, planning, engineering, personnel, and information management in the decision-making process. One of the most essential component of HAM is pavement management system (PMS) which helps the roads managers/ designers to maintain the pavements network in serviceable and efficient condition in most cost effective manner. Hudson (1968) and Finn & Hass (1977) first introduced the concept of PMS and to determine the direction and specificity of project development and planning, PMS is exercised at two levels. These two levels are PMS at network and project level (Panigraphi, 2004). Network level PMS is focused to determine and allocate funds for maintaining pavement above specified threshold to keep the pavement in operational condition. Condition forecasting, budget allocation, scheduling inspection and planning work are included at network level using prediction model. At network level most important prediction model is conducted using “what if analysis” to understand the various budget constraints for maintaining pavement condition (Shahin, 2005). Project level PMS focuses on a specific facility which need to be repaired, its method and timing for repair. Prediction models at this level are developed for selecting specific alternatives for expected climatic and traffic conditions (Shahin, 2005). An important component of most efficient PMS is its capability to predict its remaining service life (RSL) through pavement evaluation. Pavement evaluation is a generic term to determine the pavement ability to serve the users for designed life. There are two types of pavement evaluation i.e. pavement functional evaluation and structural evaluation. Pavement functional evaluation is carried out to check its ride quality through performance indicators e.g. roughness or ride quality, surface distress, rutting, skid resistance. While pavement structural evaluation (PSE) is carried out to check its structural capacity for the traffic loading for designed life and performance indicators are deflection, layer elastic modulus, subgrade CBR or resilient modulus (Irfan, et al., 2010). In past researches most PMSs focused on pavements functional evaluations, albeit functional evaluation can never be a substitute to structural evaluation, rather both accompaniment each other (Samadi, 2008). Like pavement roughness data, deflection data has also become a vital tool to evaluate the structural integrity and capacity of newly constructed or rehabilitated pavements. Pavement structural capacity reflects its load carrying capacity and normally conducted at the project level to assess whether work is needed to increase pavement strength to accommodate projected traffic loads (Irfan, et al., 2010). One of the simple and reliable method to check the structural capacity of pavement is non-destructive deflection testing (NDT) being advantageous over destructive testing that it cause little or no disruption to the traffic and its less time consuming and results are comparatively more accurate. Using NDT devices such as falling weight deflectometer (FWD) the structural adequacy of pavement is estimated by analyzing deflection basin (Flintsch, et al., 2013). In FWD total of eight deflection sensors from D₀ to D₇ measured the pavement surface deflections. The first deflection sensor D₀ is located at the center of the loading plate, while the rest seven sensors were located at varying distances up to 6 feet from the load center. In order to analyze the deflection data center-deflections are normalized and used to evaluate the pavement structure and design for overlay thickness (Noureldin, et al., 2003). Using FWD data pavement structural adequacy is determined through back-calculation process. Back-calculation approach is commonly used for pavement evaluation by calculating the layer moduli for complete pavement and also for individual layers and making most optimal pavement rehabilitation strategies (Von & Simpson, 2002). Although back-calculation is the most popular method for finding the pavement structural health, still method has four limitations observed over a period of time. First limitation is that it produce several sets of layer moduli and it’s not unique to comply with the deflection basin resulting in errors within the acceptable range (Von & Simpson, 2002). Second limitation is non-accuracy of layers moduli prediction which disturb the assumption of extending the pavement layers to infinity due to transverse & longitudinal cracking and also due to pavement edges (Uzan, 1994). If pavement layer thickness is less than 75mm, it is difficult to back-calculate the pavement layers moduli, due to the geometry of the loading and measuring system (Dynatest, 2016). Fourth limitation is AC layer temperature at the time of testing, as
layer stiffness is related to temperature variation (Park & Kim, 2002). After back-calculation of layer moduli, next important step is to predict RSL of in-service pavements. (Tonkin & Taylor, 2012). The RSL is the expected life of existing pavement in number of years during that pavement can carry the estimated traffic in condition above the threshold values both structurally and functionally (Baladi, 1991).

In Pakistan National Highways Authorities (NHA) has established Road Asset Management Division (RAMD) to maintain the road network in most efficacious manner and regulatory body known as Road Assets Management System (RAMS) Directorate part of RAMD is carrying out such activities which are needed to achieve these objectives. RAMS is using Highway Development and Management tool-4 (HDM-4) provided by World Bank as management tool to maintain its road networks. HDM-4 is computerized database which includes pavement geometric data, condition & traffic data and to some extent structural data of pavement. Database collects the pavement inventory data, pavement friction & condition data and location of pavement distresses, maintenance history of pavement, ride quality data and maintenance cost and traffic data. Compiling all this data an annual maintenance plan is prepared at national level. Annual maintenance plan includes routine maintenance plan, periodic maintenance plan and rehabilitation plan and then distributed to all the regional levels to implement on project level (Tariq, 2015).

Although RAMS partially fulfill the requirement of PMS in Pakistan but in backdrop of China-Pakistan economic corridor (CPEC) there is no effective mechanism to monitor the highway network effectively. Although collection of pavement structural data is time and resource consuming and most agencies lack in this aspect. Resultantly most network level decisions for planning rehabilitation strategies are taken on basis of pavement functional data which sometimes leads to inaccurate assessment, being muddled pavement structural health evaluation. Issues highlighted in the above paras make it necessary to carry out pavement structural evaluation of in-service pavement as integral component of PMS. Therefore there is need to develop a mechanism to integrate pavement structural evaluation into overall PMS taking pavement functional evaluation as supporting component to aid pavement structural evaluation for establishment of most optimal rehabilitation strategies. Also there is a requirement to estimate the pavement structural condition annually or at least when there is no recent structural data available. In this case NDT become most promising approach and economical tool because pavement distress is not enough to make the rehabilitation strategies. The research aims to develop a framework for integrating structural evaluation of highways in overall PMS using NDT that can be applied at network level PMS and also can be implemented at project level PMS without any difficulties.

2 METHODOLOGY FOR DEVELOPED DECISION FRAMEWORK

For rehabilitation process, a comprehensive condition survey is the most valuable tool which assist the engineer to identify the type of distress present and also lead to identify the probable cause of distress. And only by identifying the accurate cause of distress, most appropriate rehabilitation strategy can be selected. A proposed framework for integrating structural evaluation of highways in overall PMS using NDT is shown in figure 1. The stepwise procedure for developed framework including data collection, establishing pavement effectiveness analysis database, pavement structural and functional evaluation using respective input parameters, decision support tool for functional performance evaluation by determining the threshold value, calculating $S_{n_{	ext{eff}}}$ through NDT and RSL from traffic data and selection of most optimal strategy for pavement rehabilitation. Pavement functional data is analyzed by comparing it with threshold value, if violated then sections qualify for pavement structural evaluation by taking FWD data. FWD data can be analyzed either using back-calculation approach or using ELMOD software to check the structural adequacy of pavement.
2.1 Collection, Storage and Management of Pavement Data

To start with the pavement evaluation, most important consideration is to collect pavement historic data which includes pavement construction data like inventory, its maintenance & monitoring data. When pavement evaluation is carried out, data collected like visual surveys, destructive testing and NDT data are termed as bench mark data. Pavement history data confirms the reliability and accuracy of its bench mark data, like drainage data collected during pavement evaluation confirm the reliability of data from pavement historic data. 

So before conducting pavement evaluation, it is mandatory to collect both historic and bench mark data. After collecting pavement historic data, next is to collect pavement roughness data to check the ride quality and smoothness of pavement surface which must be collected annually. NDT data is collected using FWD to check the pavement structural adequacy and as normal practice it is collected biannually. There are number of sources through which data is collected but to ensure the reliability of data different quality insurance checks are performed. These checks includes correctness & calibration of equipment and personnel responsible for data collection. To ensure the quality of data, quality acceptance tools can be used for testing of data both collected by agency and by service provider. Figure 2 explains the data collection timings for pavement evaluation.
The data collected must be accurate and reliable, because all decisions pertaining to pavement rehabilitation are based on reliability and accuracy of data. After setting the rehabilitation priorities if there is any significant change it needs to be probed in detail to find out the probable cause. Once the problem is identified, most optimal rehabilitation strategy can be selected keeping in view the funds available and time to start the treatment.

3 PROPOSED FRAMEWORK APPLICABILITY: A CASE STUDY OF NEWLY REHABILITATED PAVEMENT

3.1 Evaluation of Roughness Profile of a Rehabilitated Pavement

As a case study a representative section is identified using delineation of uniform approach in southbound direction (outer lane) of M-2 Motorways to check the proposed framework applicability. Figure 3 presents the rut analysis along left and inner wheel path of section which clearly indicate that rutting along left (truck lane) wheel path is higher than inner wheel path. Figure 4 and 5 explain pavement roughness data of selected section and it is obvious that values of IRI have reduced substantially after rehabilitation from average value of IRI 4m/km to 2m/km.

Figure 3. Rut analysis along left (truck lane) and inner wheel path-south bound.

Figure 4. IRI data along outer lane before rehabilitation-south bound.

Figure 5. IRI data along outer lane after rehabilitation- south bound.

Figure 6 presents the pavement roughness data of selected section and it is obvious that values of IRI are comparatively higher on section from RD 158+000 to 182+000 and RD 241+000 to 256+000. Before commenting on IRI data of a specified section, another important aspect is to check the reliability and correctness of data which can be influenced by two main reasons i.e. profilometer is not calibrated and responsibility of crew detailed for data collection. Both concerns can be addressed through strict quality control. If reliability and correctness of data is confirmed then
these two sections are candid for structural evaluation and need to be investigated in depth by taking FWD data and analyzing the pavement structural capacity.

Figure 6. Pavement roughness profile – Outer lane (South bound) (151+000 – 259+000).

3.2 Pavement Structural Evaluation

Pavement structural capacity reflects its load-carrying capacity and normally conducted at the project level using NDT to assess whether work is needed to increase pavement strength to accommodate projected traffic loads. PSE assess the existing condition of the pavement by predicting its future performance and can be used as an integrated component of decision-making processes in PMS. It guides the pavement managers to find the most suitable pavement treatment (ranging between DOING NOTHING to complete reconstruction). PSE of existing pavement is useful for estimating its structural capacity for design life of pavement, to estimate RSL and take a decision for selection of most optimal treatment type. In this study, HMA overlay thickness design is determined using FWD data by back-calculating pavement layer moduli. It is normal to select a location of project to evaluate the overlay design parameters and this practice assists pavement engineers/managers to make decisions on choosing HMA overlay strategies. To check the applicability of the proposed framework, two sections from sec-1 (RD 158+000 to 182+000) and sec-2 (RD 241+000 to 256+000) have been identified by analyzing the IRI data. Their deflection data is presented in Figure 7 which shows that sec-1 have less deflections and doesn’t qualify for structural evaluation and only functional improvement of this section is recommended, whereas sec-2 have relatively higher deflections and are candid for further analysis to estimate the $S_{\text{eff}}$ through back-calculation or using ELMOD software.

Figure 7. Deflection ($D_0$) profile – outer lane-South bound (158+000 to 182+000 & 241+000 to 256+000).
3.3 Back-calculation of Pavement Structural Layer Modulus

Back-calculation approach is commonly used for evaluation of flexible pavements to check the structural capacity form deflection basin using FWD test, it can calculate the modulus for complete pavement and also for individual layers above subgrade level. These findings leads to make better options for redesign or rehabilitation strategies (Von & Simpson, 2002). For back-calculating layer moduli, different methods are used but basic principle is to minimize the error between measured and predicted deflections data by categorizing data into different sets basing on load analysis (Uzan, 1994). The current condition of in-service pavement plays an important role to select the overlay strategy. Although different techniques can be used to back-calculate the layer moduli but the two main purposes are; to isolate the problematic segment which need to be evaluated and to check the existing properties of the in-service pavement layer e.g. layer modulus and thickness (Irfan, et al., 2010). Structural evaluation of M-2 was carried out using HWD Dyatest 8082 using impulse load of 9430 lbs and 14,300 lbs. A total of eight deflection sensors from D₀ to D₇ measured the pavement surface deflections. The first deflection sensor D₀ was located at the center of the loading plate, while the rest seven sensors were located at varying distances up to 6 feet from the load center. The delineation of uniform sections was done using same FWD data and not using the as design and constructed sections.

3.4 Delineation of Uniform Sections.

Before starting evaluation of pavements it is mandatory to divide the whole pavement into different segments. There are two approaches to divide the pavement in uniform sections. In first approach pavement is segmented basing on design/ constructed properties using pavement historic data, in second approach delineation of uniform sections is done using criterion of change of slope of Cumulative Difference function exemplified in Appendix J of AASHTO design guide of 1993 (AASHTO, 1993). Second approach is more appropriate as it consider the existing condition of pavement rather than its design/ constructed properties which change with passage of time due to various factors. Therefore, in this study segmentation of sections was done basing on pavement deflection data using FWD.

![Figure 8. Using cumulative difference approach for Delineation of uniform sections – D₀.](image)

3.5 Calculations for Structural Number Effective (SNₐₑffective) and design for Overlay Thickness

Basic concept to design AC overlay is based on theory that structural capacity of flexible pavement can be estimated using structure number (SN). After delineation of uniform sections, pavement SN effective (SNₐₑffective) is calculated and SN future (SN_f) is calculated using future traffic. Overlay thickness design can be termed as differenc e that is going to increase SNₐₑffective of the in-service pavement to SN which is required to meet the projected traffic needs. This simple relationship to determine the SN overlay (SNₐ_overlay) can be expressed as (Huang, 2004).

\[ SN_{OL} = a_{OL}D_{OL} = SN_f - SN_{eff} \]

(1)

Where:

- \( SN_{OL} \) = Structure number for required overlay,
- \( a_{OL} \) = AC overlay structural coefficient,
- \( D_{OL} \) = Overlay thickness in inches,
- \( SN_f \) = Structure number future in terms of future traffic demand,
- \( SN_{eff} \) = Structure effective of existing pavement. \( SN_{eff} \) is determined from NDT results which are based on assumption that structural capacity of pavement is
the function of its stiffness and its total thickness. The relationship for determining $SN_{eff}$ is mentioned in AASHTO, 1993 is as:

$$SN_{eff} = 0.0045 \sqrt[3]{E_p}$$  \hspace{1cm} (2)

Where:

$E_p = $ pavement layers effective modulus above subgrade in psi. To determine $E_p$ following steps may be followed. First determine the pavement subgrade modulus ($M_R$) using equation:

$$M_R (psl) = \frac{0.24 \times P}{d_r \times r}$$  \hspace{1cm} (3)

Where:

d_r = FWD Deflection value from the center of the load in inches at a distance $r$, $P = $ Load applied by FWD in pounds, and $r = $ distance in inches from center of load plate. To determine distance $r$, basic assumption is that measured deflection must not be that far from center plate to capture the effect of subgrade and nor it should be too near to include the effect of all layers above subgrade (AASHTO, 1993). Following equation may be used to estimate distance $r$:

$$r = 0.7 \times \sqrt{a^2 + \left( \frac{D}{\sqrt{M_R}} \right)^2}$$  \hspace{1cm} (4)

Where

$a = $ Radius of load plate in inches, $D = $ Total thickness of all pavement layers above subgrade in inches. To determine $E_p$ following equation may be used and $E_p$ may be determined by trial and error using spread sheet.

$$d_0 = FWD \text{ Deflection at the center of the load plate in inches (it must be adjusted to a standard temperature of } 68^\circ F).$$

### Table 1. Statistics of Back Calculated Subgrade Moduli and $SN_{eff}$ - Southbound (241+000 to 256+000).

<table>
<thead>
<tr>
<th>Descriptive Statistics</th>
<th>Back-Calculation $M_R$ of Subgrade (psi)</th>
<th>$E_p$ (psi)</th>
<th>$SN_{eff}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>47942.6</td>
<td>70126.54</td>
<td>4.51</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>12821.41</td>
<td>23785.40</td>
<td>0.51</td>
</tr>
<tr>
<td>Minimum</td>
<td>27973.6</td>
<td>36039.14</td>
<td>3.59</td>
</tr>
<tr>
<td>Maximum</td>
<td>85434.62</td>
<td>152761.2</td>
<td>5.80</td>
</tr>
<tr>
<td>CoV</td>
<td>26.74%</td>
<td>33.91%</td>
<td>11.30%</td>
</tr>
</tbody>
</table>

3.6 Estimation of RSL based on traffic

Pavement RSL can also be predicted using traffic data, if pavement initial serviceability is $P_0$ and load repetitions are $N_t$, with increase in load repetitions and environmental effects with passage of time it decreases to $P_f$ provided no treatment or rehabilitation effort is provided (AASHTO, 1993). When serviceability is $P_f$, the number of load repetitions are $N_f$ and RSL can be expressed as;

$$RL_x = \frac{(N_t - N_f) / N_f}{N_t}$$  \hspace{1cm} (6)

Where:

$RL_x$ = The remaining life ($RL_x$) it is shown in ESALs at any time $t$.

$N_t$ = These are the number of load repetitions to reach failure.

$N_x$ = Load repetitions at any time $t$ before failure.
So pavement RSL based on traffic criteria can be defined as “Total number of load repetitions that could be applied to pavement before reaching failure point (AASHTO, 1993). Before calculating total number of load repetitions, these are converted to ESALs (18 kips). The above equation can also be written as:

\[
RL_x = 100 \left(1 - \frac{N_p}{N_f}\right)
\]

(7)

Where:

- \(RL\) = Remaining life of pavement in percentage.
- \(N_p\) = Total number of load repetitions in ESAL to date.
- \(N_f\) = Total number of load repetitions in ESAL till failure.

In Table 2, ESALs applied to pavement so far i.e. for one year \((N_p)\) has also been calculated and total ESALs for 10 years \((N_f)\) has also been estimated and details are shown in Table 4.13.

<table>
<thead>
<tr>
<th>Vehicle Type</th>
<th>AADT (2017)</th>
<th>AADT (2026)</th>
<th>NTRC Load Factor</th>
<th>Equivalent Standard axle Load (ESAL) in Million</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-Axle Truck</td>
<td>1169</td>
<td>14033</td>
<td>4.67</td>
<td>(0.80)</td>
</tr>
<tr>
<td>3-Axle Truck</td>
<td>777</td>
<td>9551</td>
<td>8.84</td>
<td>(1.01)</td>
</tr>
<tr>
<td>4-Axle Articulated</td>
<td>280</td>
<td>3436</td>
<td>10.35</td>
<td>(0.43)</td>
</tr>
<tr>
<td>5-Axle Articulated</td>
<td>94</td>
<td>1150</td>
<td>10.63</td>
<td>(0.15)</td>
</tr>
<tr>
<td>6-Axle Articulated</td>
<td>165</td>
<td>2033</td>
<td>10.9</td>
<td>(0.26)</td>
</tr>
<tr>
<td>Large Buses (Over 20 Seats)</td>
<td>1560</td>
<td>19170</td>
<td>0.94</td>
<td>(0.20)</td>
</tr>
<tr>
<td><strong>Total in Million</strong></td>
<td><strong>2.85</strong></td>
<td><strong>34.74</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Using Table 2, pavement RSL can be estimated as shown in Table 3. Although there are certain short coming of this method; inability to estimate past traffic accurately and this method doesn’t account for overlays and treatments previously applied to pavements (Shiyab, 2007).

<table>
<thead>
<tr>
<th>No</th>
<th>Section</th>
<th>KM From</th>
<th>KM To</th>
<th>ESALs (N_p)</th>
<th>ESALs (N_f)</th>
<th>RSL ((%)</th>
<th>RSL ((\text{Years}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sec 1</td>
<td>203</td>
<td>291</td>
<td>2.85</td>
<td>34.74</td>
<td>91.8</td>
<td>9.18</td>
</tr>
</tbody>
</table>

4 CONCLUSIONS

PMS aims to produce best possible pavement solutions in stipulated funds to provide safe, smooth and economical pavements. Assessment of Pavement structural capacity has become a vital component for PMS at network as well as project level. Integration of pavement structural capacity at network level is required to keep the pavement in acceptable structural condition by allocating the minimum funds. While at project level PMS it is required to guide the pavement managers to select the most optimal rehabilitation strategy at most appropriate time (varying the treatment strategy from do-nothing to complete reconstruction of pavement). Proposed framework for integrating pavement structural evaluation into PMS provides a road map to establish an effective PMS in Pakistan.

It is worth mentioning that the accurate and precise applicability and benefits of the framework can only be deliberated, judged and evaluated after the proposed enhancements of this study. The proposed methodology for project level analysis can be further augmented with development of treatment specific performance and cost models and performing lifecycle cost analysis to develop optimal maintenance and rehabilitation strategies. Serviceability observations below the acceptable level are one way to trigger a structural evaluation process. Pavement sections, if structurally not deficient may require only a functional or preventive maintenance treatment. The presented case study divulges that both functional and structural measures are important and are complementary rather than substitutes. They supplement each other, and do not replace each other, in an overall pavement evaluation.
References
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Sustainable Pavement Applications, Geogrid in Sustainable Pavement Design, Sustainable Pavement Rehabilitation, Cold In-Place Pavement Recycling, Polymer Modified Bitumen.

ABSTRACT:
As the vision of Abu Dhabi Government is to achieve Social Development in a Sustainable Manner, The Implementation of Sustainability concepts is a major objective for Municipality of Abu Dhabi City (ADM). Project Performance, Life Cycle Cost, the Use of Recycled Materials, Energy Saving are major considerations in ADM sustainable Pavement Design. Several studies were conducted by ADM team to achieve this target. In this paper, three applications of sustainable pavement concept in design and rehabilitation of asphalt roads in Abu Dhabi are presented.

The first application is the use of Geo-grid in sustainable pavement design. In this case study, the Geo-grid is used for reinforcement of granular base course, or subbase. Design analysis was conducted using AASHTO1993 method, and the contribution of Geo-grid material is characterized by the Layer Coefficient Ration (LCR), which is based on field and laboratory testing as per AASHTO – R50 – 09 standards. This application was implemented during last 6 years, resulted in several achievements as; longer service life of pavement up to 50%, reduction of pavement thickness, Reduction of Pavement life – cycle cost (up to 120 One Hundred and Twenty Million AED), Reduction of Construction time about 20%, Saving in Row Material and Energy about 10%, and Reduce of Carbon emissions about 15%.

The second application is Utilizing Cold In-Situ Pavement Recycling in Rehabilitation of life ended asphalt pavement. The main objective of this application is to utilize the recycled pavement material by adding foamed bitumen to extend pavement service life and reduce construction cost. A pilot project was conducted in 2011, and design analysis for several projects in 2014. Several benefits were achieved by this application as; extending pavement service life, construction cost reduction by 30%, saving in raw materials of 80%, reduction in energy consumption of about 40%.

The third application is the use of Polymer Modified Bitumen (PMB) in construction of intersections at Mussafah Industrial Area, Abu Dhabi. The main target of this application is to have more durable asphalt pavement at intersections of industrial areas, which subjected to higher volume of heavy trucks, with low speed and breaking force at intersections. The use of PMB enhanced pavement performance and reduced rutting rapidly, compared with conventional asphalt application.

Technical, financial and environmental assessment and ADM acceptance criteria of the three applications are addressed in this paper.
Application of Sustainable Pavement Concepts in Design and Rehabilitation of Abu Dhabi Road Projects

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1 INTRODUCTION

Municipality of Abu Dhabi City (ADM) has a strategic objective to implement sustainability principals in roads and infrastructure projects. To achieve this target, Infrastructure Sustainability Team (IST) had been established by the Infrastructure and Municipal Assets sector since 2011, to conduct relevant application research studies, and to implement in roads and infrastructure projects. For Road Projects, Several application research studies were conducted by ADM - IST along the last six years; Studying the use of Geo-grids for Mechanical Stabilization and reinforcement of granular pavement layers as sustainable pavement design technique, Application of Cold In-Place Asphalt Pavement Recycling in sustainable pavement rehabilitation, the use of crushed recycled concrete aggregate as sustainable granular base and/or sub-base course layers in construction of Abu Dhabi Road Pavements, and the application of polymer modified bitumen in asphalt mixes at heavy traffic intersections.

In this paper, three applications of sustainable pavement concepts in design and rehabilitation of Abu Dhabi road projects, conducted by ADM team, are presented; Sustainable Pavement Design Utilizing Geo-grids, The use of cold in – place recycling in sustainable pavement rehabilitation, and Application of polymer modified bitumen at Abu Dhabi intersections. The main objective of these three applications is to achieve more sustainable pavement by increasing pavement service life, enhancing pavement performance, decreasing life – cycle cost, decreasing maintenance rate, and saving environment by decreasing carbon emissions, energy consumption and the use of raw materials.

ADM team implemented a comprehensive methodology in studying, applying and specifying different sustainable applications in Abu Dhabi road projects as follows:-

1. Studying the nature, history, previous applications & testing results, design calculations, cost analysis, environmental impact assessment.
2. Evaluating the possibility of application in Abu Dhabi road pavement, considering; material characteristics, design calculations, availability in Abu Dhabi (or UAE market), suitability to Abu Dhabi relevant specifications and environment, constructability and required maintenance activities,
3. Quantification of expected benefits from technical, financial and environmental points of view, considering; extending pavement service life, less life – cycle cost, minimized carbon emissions and saving natural environment, reducing the use of raw materials, saving water resources and energy consumption, and reduced construction time, when compared with conventional practice,
4. Conducting a pilot project, as suitable, to test suitability to Abu Dhabi environment and conditions, from the practical point of view,
5. Monitoring the constructed pilot project, up to an agreed time duration,
6. The whole study is recorded in a brief report. Testing results, quantified benefits; technical, financial and environmental, and the recommendation to apply this material (or technology) in Abu Dhabi Emirate Road Pavements, to be included in the report,
7. Communicate with relevant Abu Dhabi authorities, with the study report, attached with a proposed draft specification (or acceptance criteria), to be studied within relevant governmental committee, and then to be included in Abu Dhabi relevant Specification.

2 SUSTAINABLE PAVEMENT DESIGN UTILIZING GEO-GRIDS

2.1 Introduction and Background

The main objective of utilizing geo-grid in pavement design is to; achieve more sustainable pavement by extending pavement service life (and/or) reduction of pavement layer thickness, enhance pavement performance under different service conditions, reduce life – cycle cost and minimize negative impact on natural environment by reducing carbon emissions during construction activities. The mechanism of geo-grid contribution in pavement structure can be summarized in two major actions; Lateral confinement of aggregate particles, which increasing load carrying capacity
and improving pavement structural performance, and increased load spreading over a wider area as the key mechanism for pavement reinforcement.

Several successful case studies of introducing geo-grid in reinforcement of road pavement layers, were experienced internationally. The following are some project cases;

A. The A1073 project, Lincolnshire, UK in 2009 created 22 km of new highway. Conventional design have required 0.5−1 m of ground replacement across the full width of the carriageway. This would have generated serious environmental, cost and contractor’s program issues. As a value-engineered alternative to subgrade replacement for when the project encountered weak ground conditions, a mechanically stabilized layer comprising a geogrid and. This saved between 0.5 and 1 m of subgrade replacement and reduced the pavement thickness and construction cost and time. It was concluded that mechanically stabilized layers have been used in several projects in the UK, taking advantage of the analysis [7].

B. In 2009, a road project for Missouri DoT-USA, was constructed and resulted in significant benefits for the project as; asphalt thickness reduction of 2 inches, increased Equivalent Single Axle Loads - ESALS of > 20%, and cost saving of asphalt of 20%, with achieving the same pavement service life. Other road project were constructed at 2009 for the Town of Harrison, New York-USA. The application of Geo-grid in sustainable pavement resulted in significant benefits for the project as; asphalt thickness reduction of 2 inches and cost saving of asphalt of 15%. Figures 1 & 2, showing photos during construction of these two projects.

Figure 1. Missouri DoT Road Project – 2009  
Figure 2. New York Road Project - 2009

2.2 Design Approach

ADM team have followed the following steps to conduct/review pavement design, conducted by consultant(s), utilizing geo-grid material as reinforcement for road base course and road sub-base, with reference to [2] and its references;

- Technical review of material properties, certification and previous implementation in similar cases,
- Verification of material properties by recognized third party lab certification,
- Review of pavement design criteria, referring to [1],
- Review of the derivation methodology of design parameters, expressing contribution of geo-grid material in overall pavement structural number as per AASHTO-1993, referring to [13&14&6],
- Review of the independent - third – party report, addressing the review of pavement design criteria and verification of design parameters,
- In case of using an In – House design software, verification of pavement design analysis, using different calculation tool is conducted,
- Conducting AASHTO layered analysis to check minimum required layer thickness based on the value of resilient modulus M_r,
- Considering existing and future utility lines and expected future maintenance of road and infrastructures with existence of geo-grids,
- Evaluate design options (conventional & Geogrid designs) (technically, financially and environmentally) by assessing the expected service life, calculating life cycle cost and carbon emissions respectively, and
- Selection of most suitable design option accordingly.

2.3 Sample ADM Applications

The following are two sample case studies, addressing implementation of geo-grid reinforcement of road base course in Abu Dhabi Road Projects, referring to Ref. [2], in order to achieve more sustainable pavement sections, highlighting technical, financial and environmental benefits achieved by applying this methodology.
A. Case Study #1: Pavement Design of Industrial Project – Main Road

This road was designed at 2011, as a main road at industrial area in Abu Dhabi, to serve about 134,000,000 ESALS. The implementation of geo-grid material in pavement design resulted in significant benefits for the project as:-

- Technically; Enhancement of granular base course performance and reduction of pavement thickness,
- Financially, Cost saving of AED 2,000,000 (about 550,000,000$) (about 17% of pavement cost), and
- Environmentally, Carbon foot print reduction of about 15%. Figure 3 shows pavement cross sections of conventional & geo-grid design and road construction with geo-grid.

![Conventional Design vs Geo-grid Design](image)

**Figure 3.** Pavement cross sections of conventional & geo-grid design and road construction

B. Case Study #2: Pavement Design of Residential - Commercial Project

This road was designed in 2014, as an Urban - Arterial road at residential-commercial area in Abu Dhabi, to serve about 28,000,000 ESALS. The implementation of geo-grid material in pavement design resulted in significant benefits for the project as:-

- Technically; enhancement of granular base course performance and reduction of pavement thickness,
- Financially, cost saving of AED 41,800,000 for all project roads (about 11,500,000$) (about 14% of pavement cost),
- Environmentally; carbon foot print reduction = 18%, Figure 2 shows pavement cross sections of conventional & geo-grid design. Figure 4 shows the conventional and geo-grid design options.

![Conventional Design vs Geo-grid Design](image)

**Figure 4.** Pavement cross sections of conventional & geo-grid design
2.4 Findings and Achievements

A. Technical Achievements

ADM case studies showed several technical achievements; Enhance mechanical characteristics of reinforced road base course/sub-base (CBR, Mr.,...), provide longer service life for pavement and (or) reduce the required pavement thickness, reduction of asphalt pavement distresses; (structural rutting, fatigue cracking,...), and reduce the risk related to construction quality of pavement.

B. Financial Achievements

Reduction of pavement construction and life–cycle costs of about (10 – 20) % with an overall achieved cost reduction of road pavement works of Abu Dhabi projects, designed with geo-grid materials of more than 120 million AED (about 32,500,000$) (One Hundred and Twenty Million Emirates Dirhams), reduction of construction time of pavement works of about (10 – 15) % and saving in energy consumption during construction,

C. Environmental Achievements

Reduction of the use of raw materials (10 – 20)%, Reducing carbon emissions during material production, transportation and construction, Decreasing number of trucks required for construction, and reducing air pollution and noise during construction, resulting in lower impact on public health.

2.5 Acceptance Criteria For Design of Flexible Pavements, Reinforced with Geo-Grids

In order to standardize material and design requirements for utilizing geo-grids in reinforcement of granular pavement layers of Abu Dhabi roads, ADM team developed the first revision of “ADM Acceptance Criteria of Flexible Pavement Design Utilizing Geo-grids”, in June 2012, Including; Material properties verification, including updated certification of material properties, and pervious applications in similar project cases, and General design process, including design considerations, reference standards, design steps and design requirements. Acceptance criteria was updated in 2014 and included in Abu Dhabi Emirate Pavement Design Manual - 2017 [2]. For more details of Abu Dhabi Emirate Acceptance Criteria, see Ref. [2]-Annex 3.

3 THE USE OF COLD IN-PLACE RECYCLING IN SUSTAINABLE PAVEMENT REHABILITATION

3.1 Introduction and Background

Recycling is defined as “The reuse, usually after some processing, of a material that already has served its first-intended purpose”. Relative to asphalt pavement recycling, there are several methods available. Therefore, each project being considered for recycling must be carefully evaluated to determine the most appropriate method. The factors to be studied for each project should include: Existing pavement condition, Existing pavement material types and thickness, Recycled pavement future structural requirements and Availability of recycling technique and additives. [5]

Recycling can be broadly classified as central plant recycling and in-situ recycling. If the RAP is modified at a plant, away from construction site then the process is known as central plant recycling. At In-situ recycling process the RAP modified in place, where from it is available. Further, the RAP could be heated to condition it. If heat is applied then the process is known as hot mix recycling. In case of cold mix recycling, old materials are conditioned using recycling agent (like low viscosity emulsion) without application of heat. [3]

Pavement Recycling with Foamed asphalt is not a new concept with respect to pavement rehabilitation and stabilization. Many countries in the world use the process as a tool to help maintain the integrity of their transportation infrastructure. Some states in the US have already used asphalt pavement recycling with success. This information was then used to develop a design guideline and specification for the use of pavement recycling with foamed asphalt in road Minnesota. [10]

Bituminous pavement recycling technology is not yet popular in Abu Dhabi. However, in other countries, bituminous material is the most recycled material in the construction industry. For example, in USA, 100 million tons of Recycled Asphalt Pavement (RAP) is used per year for recycling purpose which is around 95% of the total amount of RAP collected from old bituminous pavements. The amount of RAP used for recycling per year is about 0.84 million tons in Sweden, 7.3 million tons in Germany, 0.53 million tons in Denmark and around 0.12 million tons in Netherlands. In the year 1995, 20 million tons of recycled hot mix was produced in Japan, which constituted 30% of the total hot mix production. [8]
3.2 Study and Pilot Project Approach

Based on the factors; Existing pavement condition, Existing pavement layers material and thickness, Future Requirements of recycled pavement and Availability of recycling technique and additives, the technique of Cold In-Place recycling was selected to be studies versus the conventional technique utilized for similar cases. The cold recycled material was considered to be classified as Stabilized Road Base Course, consists of; Recycled Asphalt Mix + Recycled Sub-base Course + Cement and Foamed Asphalt Additives. The Methodology of Pilot Project Study includes the following:-

1. Evaluation of Existing Road Pavement condition, and selection of most suitable road maintenance technique,
2. Material Characterization of Existing Pavement Layers and Sub-grade Soil,
3. Mix Design and Material Characterization of Cold In-Place Recycled Layer,
4. Analysis of rehabilitation design options (Recycling option versus conventional technique),
5. Field implementation and post recycled pavement testing,
6. Evaluation study, comparing Cold In-Place pavement Recycling with Conventional Rehabilitation Technique (Technically, Financially and Social Environmentally),
7. Summary of Results, Conclusions and Recommendations, and
8. Monitoring of Reclaimed road, to assess pavement performance against rehabilitation design.

3.3 Implementation of Pilot Project

A pilot project study was conducted in 2011, to study Cold In-Place Recycling as a design option compared with conventional pavement rehabilitation by removal and reconstruction of Road Number (31/16) in Shahama area as a part of the project “Study, Evaluation, Design & Supervision of Road Maintenance & Rehabilitation Works of Abu Dhabi Main Island – North”. A comprehensive study was conducted by ADM team, with the following steps;

A. Evaluation of Existing Pavement Condition of North Road (31/16),

Block cracking, Asphalt hardening, were observed within the (31/16) road. By inspection of existing road pavement, deep block cracking (≥ 10 cm.) were observed. It was concluded to remove of existing asphalt of 15 cm. then checking the existing road sub-base, and reconstruct the pavement in addition to study Cold In-Place Recycling as an alternative option.

B. Material Characterization of Existing Pavement Layers and Sub-grade Soil,

Test pits of (80*80*80-100) cm. were excavated. Samples were taken from each pavement layer separately to be tested accordingly. Test pits showed that existing pavement layers are; 5 cm. of Asphalt Wearing Course, 10 cm. of Asphalt Base Course, 15 cm. of Road Sub-base Course, and 50 cm. of compacted Sub-grade Soil. Gradation, Liquid limit and Plasticity Index, Proctor and CBR tests were conducted on sub-grade soil and road sub-base, while Extraction was conducted. For or more details of testing results, see ref. [12]

C. Mix Design and Material Characterization of Cold In-Place Recycled Layer

It was studied to mill 5 cm. of existing wearing course, recycling of 10 cm. of Asphalt Base Course + 10 cm. of existing road Sub-base, and a new overlay of 5 cm. Asphalt Wearing Course is to be constructed. Proportioning of mix design of recycled layer (Stabilized Road Base Course) is as follows:-

- 96% of Recycled Material (Asphalt + Road Sub-base),
- 1.5% of Normal Cement, and
- 2.5% of Foamed Bitumen (97.5% Bitumen + 2.5% Steam).

Material Characterization of recycled mix was conducted utilizing Marshall Test in addition to Indirect Tensile Strength for dry & immersed samples for 24 hours. Testing indicated that Marshall Stability of recycled material is almost same as Asphalt Base Course, Excellent strength and excellent strength retention after immersion of 24 hours in water of 25°C, which resulted in higher value of correlated layer structural coefficient (0.11/cm.), leading to expected Reduction in overall re-designed pavement thickness. For or more details of testing results, see ref. [12]

D. Analysis of Rehabilitation Design Options,

Design analysis was conducted, referring to AASHTO-1993 standard, to study both rehabilitation options; removal and re-construction of Asphalt layers + Road sub-base, and to implement Cold In-Place Recycling of 10 cm. of Asphalt Base Course (ABC) + 10 cm. of Road Sub-base. Design analysis was referred to Abu Dhabi Municipality Guidelines, 2012, in addition to Abu Dhabi Road Design Manual, 1998. Design life was specified to be 20 years. All design inputs were the same in both options, except pavement material properties. Design analysis results showed that
conventional pavement section of (5 cm. Asphalt Wearing Course + 10 cm. of Asphalt Base Course + 15 cm. of Road Sub-Base) is equivalent to the recycled pavement section of (5 cm. Asphalt Wearing Course + 20 cm. of Recycled Stabilized Base Course + 5 cm. of Road Sub-Base). Pavement design calculations are addressed in reference ref. [12].

E. Field Implementation,

Execution was started by removal of existing wearing course of 5 cm, then recycling started by levels and slopes adjustment and specifying recycled thickness. During recycling process, the additive (cement and foamed bitumen) percentages and recycling depth were continuously monitored. Samples of recycled material were taken from different locations for Quality control and Assurance. Once the recycling process is finished, compactors were utilized to compact the recycled stabilized base course. After compaction, the road was opened to traffic by several (3 - 7) days for curing (aeration to evaporate water from recycled layer) to ensure full compaction and hardness of recycled layer. After a week from recycling is completed, the new asphalt wearing course of 5cm was laid.

Figure 5. Construction of Shahama cold in-place pilot project

3.4 Evaluation of Pilot Project Results

Results of pilot project were evaluated from technical, financial and Environmental points of view as follows;

A. Technical Benefits,

Cold In-Place recycling resulted in Increasing Rigidity of Stabilized Road Base Course, which resulted in providing stronger support to asphalt wearing layer, leading to prevent structural rutting and fatigue cracking. In addition, the recycled stabilized layer reduces moisture susceptibility of ground water to asphalt. The layer coefficient of road base course (or subbase) is increased from 0.05/cm to 0.11/cm, leading to more sustainable pavement design by decreasing the required thickness of road base course, (and / or) increasing pavement service life with the same layer thickness.

B. Financial Benefits,

Comparison was made between conventional rehabilitation method, and the second option of cold in-place recycling, for time, cost and energy consumption for each construction activity for both rehabilitation options. Comparison resulted in reduction of construction time and cost for recycling process than conventional rehabilitation by 75%, 30% and 40% respectively. This significant reduction in time, cost and energy consumption is referring to reduction of raw material production, transfer and construction for road base course material, raw materials (aggregate + bitumen) for asphalt base course. In addition, construction time of recycling is greatly reduced by the construction technique itself, with rate of about 285 – 300 m²/hour.

C. Environmental Benefits,

Cold in-place recycling process resulted in several benefits for Abu Dhabi environment as follows; 100% of the Existing Road Materials are Re-Utilized, Resulting in Reduction in Waste Generation, Reduction of the Use of Virgin Materials of 80%, and Re-Use of Binder Asphalt of Recycled Asphalt Pavement, Savings in Energy for Heating, Transportation and Construction of about 40%, Reducing Air Pollution and Noise during Construction, Resulting in Lower Impact on Public Health, Less Traffic Delay due to Traffic Detours, and minimized Emissions from Delayed Traffic, Trucks Needed for Construction, are Lowered by 80%, and as a Result, About 83% Reduction in Carbon Emissions.
3.5 Application of Pavement Recycling in Design of ADM Projects

Cold In-Place Recycling was studied as a design option for the project “Pavement Rehabilitation of Road Network at KHALIFA City (A), 2014 - Abu Dhabi”. Two design optioned were conducted, considering conventional rehabilitation method by removal and reconstruction of the existing pavement, against application of cold in-place pavement recycling with foamed bitumen & cement. Several significant benefits were achieved. Technically; Decreased the required pavement thickness of about 23%, with ensuring same design life, and increased resistance to ground water negative effect on pavement structure. Financially; Construction cost reduction of 41,000,000 AED (about 11,000,000$) (about 18%), faster construction by 40%, and less energy consumption of about 30%, and Environmentally; 80% of existing road materials are re-utilized, and 60% reducing of carbon emission and noise during construction, resulting in Lower Impact on public health.

3.6 Abu Dhabi Specification of Cold In-Place Asphalt Recycling

ADM team issued the first edition of ADM Specifications for Cold In-Place Recycling using Foamed Bitumen & Normal Cement at 2012, in order to start implementation for suitable projects. Specification included Scope and References, Component Material Requirements (Aggregate, Foamed Bitumen Binder, Cement, and Required Quality of Water), Mix Design Requirements for Recycled Layer, and Construction Procedure & Equipment. For more details, see ref. [11].

4 APPLICATION OF POLYMER MODIFIED BITUMEN AT ABUDHABI INTERSECTIONS

4.1 Introduction

Flexible pavement needs to be flexible enough at low service temperatures to prevent pavement cracking and to be stiff enough at high service temperature to prevent rutting. Bitumen modified with polymer offers a combination of performance related benefits as the physical properties of the bitumen is improved without changing the chemical nature of it. [4]

Polymer modification was found to be among the most commonly used binder modifications. In general, two types of polymers are used to modify bitumen for road construction: plastomers and elastomers. Examples of commonly used plastomers are Ethene-Vinyl-Acetate (EVA) and Polyethylene, while Styrene-Butadiene-Styrene (SBS) is the most used elastomer and probably the most appropriate polymer for bitumen modification. Basically, plastomers increase the stiffness and viscosity of bitumen while elastomers improve the elastic behavior of bitumen [14].

Several Studies have revealed that properties of bitumen and bituminous mixes can be improved/modified with addition of certain additives and the bitumen premixed with these additives/modifiers is known as “modified bitumen”. Studies have shown that permanent deformation (Rutting) within flexible pavement is usually confined to the top 100 to 150 mm of the pavement also known as surface course [9].

As traffic grows in Abu Dhabi due to development projects, especially heavy traffic at industrial areas, asphalt rutting was observed constantly at a signalized intersections in Musaffah Industrial Area. As traffic congestion takes place on regular and daily basis by heavy truck vehicles as it is breaking at traffic signals with heavy load, in addition to high service temperature most of the year, Rutting is more likely to be found.

ADM maintenance team proposed to study implementation of a different solution to reduce the high rate of maintaining asphalt pavement at intersection M7 at Musaffah Industrial Area, which more than 10 cm of asphalt rutting was observed. Based on that, ADM technical team conducted a study, in cooperation with maintenance team, to use the SBS polymer modified bitumen in replacing the existing rutted asphalt to improve asphalt performance to resist rutting.

4.2 Study and Pilot Project Approach

As the SBS polymer is the most used elastomer and probably the most appropriate polymer for bitumen modification, in addition to availability in UAE, It was selected to be used as a polymer modifier for replaced asphalt base course and wearing course. The RWplast®M, is a granule composed of Plastomer and filler which is designed to increase the resistance of base/binder course at high traffic zones, heavy and channelized traffic, high load bearing area and slow speed, While RWelast®E provides Better resistance to permanent deformation, an increase in the road pavement lifespan, and a reduction in maintenance operations of wearing course. The following steps where followed in conducting the study and pilot project;

1. Evaluation of Existing Road Pavement condition,
2. Conduct detailed review of characteristics of asphalt mix performance of Polymer Modified Bitumen (PMB),
3. Mix Design and performance characterization,
4. Analysis of rehabilitation design options (PMB versus conventional technique),
5. Field implementation of Four Lanes of 125m. Length,
6. Evaluation study, assessing benefits of PMB compared with Conventional maintenance technique from technical and financial points of view.
7. Summary of Results, Conclusions and Recommendations, and
8. Monitoring of constructed pilot project,
9. Producing a draft specification of PMB (Rutting Resistance Asphalt Mixes) to Abu Dhabi relevant authorities,
   to include in Abu Dhabi emirate relevant specifications.

4.3 Implementation of Pilot Project

Pilot project was conducted in March, 2015 to implement PMB at Intersection M7 in Musaffah Industrial area,
Abu Dhabi as follows;

A. Evaluation of Existing Pavement Condition,

Intersection M7 at Musaffah Industrial Area consists of 1 right turn lane, 2 through lanes, and 1 shared lane for
through and left turn, in addition to a shared lane of left and U-turn. Visual inspection showed medium to sever rutting
at the left lane, ranging from 5cm. to 11 cm. and Sever rutting at the mid three lanes (10 – 11) cm. and medium rutting
at the right turn lane.

B. Mix Design of Polymer Modified Asphalt Base Course and Wearing Course,

Mix design of asphalt base course and wearing course was conducted under the supervision of polymer
supplier and ADM Material Quality Section, as per ADM standard specifications. The polymer RWplast®M was added
with a percentage of 0.30% of the asphalt base course mix, and the RWelast®E was added with a percentage of 0.57%
of the wearing course mix. Asphalt 60/70 was used for both asphalt mixes.

C. Laboratory Testing of Polymer Modified Asphalt Mixes Design,

Mix design procedure for aggregates, bitumen and RWelast®E and RWplast®M granules for producing
asphalts in the laboratory was conducted as per the "Protocol for making asphalts in the laboratory with RWelast®E
solution" (see appendix 1 of Ref. [15]). Several parameters were considered as affecting rut resistance in mix design;
the first is binder content not to be high and grade not to be too soft, the second is the dosage of filler and sand not to be
high, the third is the voids content to be in mid-range. Laboratory mix design was conducted as per European Standard
(EN 12697-35), rutting test as per (EN 12697-22) and Asphalt Pavement Analyzer APA (AASHTO T 63), (see Ref.
[16].

D. Execution of Pilot Project,

The distorted area has milled to depth of 12cm, the wasted material has disposed properly away from the site,
and surface has cleaned, Tack coat was applied and protected for 2hrs, Modifier product stored properly and transported
to the asphalt plant as doses, The polymer was mixed with the aggregates before adding the bitumen under supervising
of ADM Material Quality Section, The asphalt base course has transported to site, spread and compacted in a proper
way, Wearing Course has spread as per ADM standard specification, Ceramic studs have been used as a road marking
to finalize the surface and Curb stone has repainted as finalize for the works of the asphalt.

4.4 Evaluation of Pilot Project Results

The use of PMB with asphalt mixes of asphalt base course and wearing course of Musaffah Industrial M7
intersection resulted in significant technical and financial benefits; Better resistance to permanent deformation, an
increase in the road pavement lifespan, and reduction in maintenance operations. An Increase in asphalt layer cost of
13.9%/m², corresponding to extending in pavement service life from 8 months to 2.5 years (3 times service time
extension), was experienced.

4.5 Abu Dhabi Specification of Rut Resistance Polymer Modified Asphalt Mixes

ADM team issued the first edition of ADM Specifications for Rut Resistance Polymer Modified Asphalt Mixes
at 2016, to start implementation in projects. Specification includes References, Material Requirements (Aggregate,
Filler, Bitumen, Polymer, and Required Quality Control, Mix Design Requirements, and Construction Procedure are
addressed. For more details for the specification, see ref. [16].
1. Achieving sustainable roads & infrastructures is one of ADM major objectives,


3. Longer service life, less LCC, better serviceability, easier construction and maintenance, saving in raw materials, water and energy, less carbon emissions are basic considerations in pavement design for ADM projects,

4. Application of geo-grid in mechanical stabilization and reinforcement of road pavement is considered as one of sustainable pavement design options, and resulted in significant technical, financial and environmental benefits,

5. Pavement recycling approach is widely implemented over more than 30 years, as a major sustainable pavement rehabilitation approach and provided more durable road base course for rehabilitated Pavement,

6. Pavement Recycling Pilot project study and designed projects, resulted in reduction of cost, materials, energy and carbon emissions,

7. Application of PMB Asphalt mixes at heavy traffic intersections of Industrial city resulted in enhancement of rutting resistance and sustaining heavy traffic loading, and increased service life,

8. Sustainable pavement design with geo-grid, pavement recycling and PMB asphalt mixes Approaches are Complying with Abu Dhabi Government Direction for Implementation of Sustainability Principals in Roads & Infrastructure Projects.

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END OF PAPER
PAPER TITLE: FINDING WAYS TO MAXIMIZE RECYCLING OF RECLAIMED ASPHALT PAVEMENT IN A HIGHWAY PROJECT

TRACK: Sustainable Pavement Design & Management

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ABSTRACT:
It’s since almost a decade that Sacyr faced the challenge of reducing or eliminating waste material generated when distressed asphalt pavement is removed. SACYR is convinced about the opportunity that those materials represent to improve existing technology, to implement sustainable development principles and to reuse a technically and economically valuable material. Starting from the knowledge of the construction company on asphalt concrete design and production technology and the growing demand of long-term pavement maintenance management of the concession company, the opportunity of joining forces together for facing these challenges arose and a wide variety of R&D projects where proposed and developed in recent years. Some of these important studies came out in a real production scenario during the construction and rehabilitation of Autovía del Arlanzón concession, a project including 1,402,157 tons of asphalt hot mix and a huge amount of RAP to be generated with a potential use in the new mix itself. In the ongoing Sacyr commitment for eliminating waste and by-product materials, it was found appropriate to implement recycling technologies, optimizing generated RAP reuse, with different recycling methods produced at SACYR CIRCULAR RECYCLING ASPHALT PLANT.
Finding Ways To Maximize Recycling Of Reclaimed Asphalt Pavement In A Highway Project

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1 INTRODUCTION

Recycling reclaimed asphalt pavement (RAP) is a valuable approach for technical, economical, and environmental reasons. The use of RAP should be favored over virgin materials in the light of the scarcity of quality aggregates, specially in some countries in the Middle East region, and the pressuring need to account for the environment. Many countries have reported significant savings when RAP is used.

The use of RAP also decreases the amount of waste produced and helps to resolve the disposal problems of highway construction materials, a problem well known and extended in Middle East. It’s been 30 years since the beginning of use of RAP as a material for asphalt mix and it appears that the use of RAP will not only is an actual alternative but it will be a real necessity in the future to ensure economic competitiveness of flexible pavement construction.

It’s since almost a decade that Sacyr faced the challenge of reducing or eliminating waste material generated when distressed asphalt pavement is removed. SACYR is convinced about the opportunity that those materials represent to improve existing technology, to implement sustainable development principles and to reuse a technically and economically valuable material. Starting from the knowledge of the construction company on asphalt concrete design and production technology and the growing demand of long-term pavement maintenance management of the concession company, the opportunity of joining forces together for facing these challenges arose and a wide variety of R&D projects where proposed and developed in recent years. Some of these important studies came out in a real production scenario during the construction and rehabilitation of Autovía del Arlanzón concession, a project including 1.402.157 tons of asphalt hot mix and a huge amount of RAP to be generated with a potential use in the new mix itself.

In the ongoing Sacyr commitment for eliminating waste and by-product materials, it was found appropriate to implement recycling technologies, optimizing generated RAP reuse, with different recycling methods produced at SACYR CIRCULAR RECYCLING ASPHALT PLANT. This paper describes this project as a paramount example of the wide variety of recycling techniques that may be applied if adequate plant technology is available.

2 ARLANZÓN HIGHWAY PPP PROJECT

The concessional contract “Maintenance & Management for highway A-1 kilometres points 101 to 247 (Segovia and Burgos provinces), was awarded by the Spanish Ministry of Public Works in November 2007. This contract was included within the "First Generation Highway Conditioning Plan” of the Ministry.

In a brief resume, Highway Conditioning Plan has the purpose to update the design parameters based on the road alignment improvement and a full width overlay of the existing pavement, at the six principal road routes in Spain. This contract includes the following activities during validity term (19 years):

1. Initial phase: reform and modernization of the highway. In this phase, the following actions have been carried out (3 years):
   • Renewal of pavement, drainage, slopes in 216 km of road (108 km center lanes)
   • New road construction, five sections of new dual carriageway (13 km total)
   • Existing road update to current regulations (alignment, layout, equipment, drainage, slopes) in sections that total a total of 50 km of roadway (25 km of trunk). An example of this kind is in figure 1

2. Operation phase: maintenance and exploitation of the infrastructure from the contract beginning, including replacement and major repairs and ordinary maintenance.
1 CIRCULAR RECYCLING ASPHALT PLANT

With the facilities installed for the Arlanzón Highway, Sacyr has ability for address any kind of RAP reused or recycling technology. The core machine is a INTRAME 260 batch plant complemented with other facilities described below.

Facility nº1: Hot Mix Asphalt batch plant with pug-mill recycling technique
- Counter flow aggregates dryer drum plant (260 ton/h production rate)
- 5 cold feed bins for virgin aggregates
- Emission control bag-house
- Hot screens and mixer (3250 kg) tower
- 2 bitumen tanks 90tons each

Facility nº2: Hot Mix Asphalt batch plant with pug-mill recycling technique
- Counter flow aggregates dryer drum plant (260 ton/h production rate)
- 2 RAP Cold feed bins + 2 independent RAP weigh hoppers
- Independent conveyed RAP LINE entrance to the RAP-DRUM
- separate RAP weigh hopper feeding directly to the pug-mill
- steam release system from the pug-mill
- RAP bypass for low RAP percentage asphalt mixes

Facility nº3: Half Warm Mix Asphalt batch plant with secondary RAP drier drum recycling technique
- RAP-DRUM, Parallel flow convective heat transfer RAP dryer, 130 ton/h (Outer back placed combustion chamber)
- Bucket elevator for heated RAP
- separate RAP weigh hopper feeding directly to the pug-mill
- Emulsion storage and feeding facility directly to the pug-mill

Facility nº4: In line RAP breaker and screen deck
- In line roller crusher (lump-breaker)
- Screen deck for fractionating RAP two sizes [(3/16” (5 mm) & 7/8”(22mm)]
- 2 RAP Cold feed bins + 2 independent RAP weigh hoppers
- 2 different conveyed entrances to the RAP-DRUM
- Additional burned gases recirculation
3 RECYCLING TECHNIQUES

The Arlanzón Highway was the perfect situation for developing most advance recycling technologies at Sacyr. On one hand, one of the main actions to accomplish was producing 1.4 million tons of Hot Mix Asphalt. On the other hand, a large amount of milling material from the existing road platform. For this reason, it was decided to promote RAP (Reclaim Asphalt Pavement) technologies. This action focuses at the p SACYR’s corporate social responsibility goal of reducing construction waste.

For this purpose, in addition to studies on recycled mixes with RAP, Sacyr Construction proceeded to accelerate and complete some of the developments still underway with new investments to give a satisfactory answer to the problem. Therefore, three different solutions have been developed using SACYR CIRCULAR RECYCLING ASPHALT PLANT:

1. Used of milled RAP as aggregate in Hot Mix Asphalt (15% RAP)
2. Recycling milled RAP with Hot Mix Asphalt technologies (40% RAP)
3. Recycling milled RAP Half Warm Asphalt technologies, experimental section (100% RAP)

3.1 Used of milled RAP as aggregate in Hot Mix Asphalt (15% RAP)

It consists of using the milled material as an additional aggregate fraction up to a maximum of 15% by weight of the mix. In this work the milling has been added up to a proportion of 10% (maximum in normative until 2014).

In this solution, the proportion of binder that is contributed by milling is small and is not consider either for the dosage or for the characteristics that the new binder must meet. No additional requirements are established for mixtures design, just the same tests and perfroms as for any other new mix. In manufacturing process, the milled material is simply subjected to a screening process to remove the larger sizes and is fed without heating directly to the plant mixer.

It is the simplest solution found at PG-3 (General Specification from Ministry of Public Works) article 542 of. Up to 15% of RAP is allowed, considering this milled material (aggregates plus aged bitumen) just like aggregates in any other mix. Any layer and any traffic category is allowed except for wearing course. In this case, this solution has been applied in the intermediate and base layer of the highway. With this technology 48,600 t of milled material have been consumed.

Sacyr Circular Recycling Asphalt Plant for this technique, process milling material (RAP) in real time, allowing continuous control and homogenization. The manufacturing process is constituted by (figure 3):

- Disintegration of milling in a specific mill
- Classification in two different fractions
- Continuous weight measurement of each fraction
- Bypass for cold RAP conveyed to the bucket elevator
- Introduce RAP, virgin aggregates, bitumen and filler at the pug-mill for batch mixing
The manufacture of fine fraction is also done at the time of consumption to avoid the problems of its collection (agglomeration and accumulation of moisture). All these operations are integrated in a mobile module. Subsequently the fractions of the milled material are added to the mixer along with the rest of the mixture materials (virgin aggregates and new binder).

When cold RAP is added to the weigh hopper where the batch controls weigh it as an additional material. The RAP is mixed with super-heated virgin materials, and conductive heat transfer occurs in the pug-mill. With the additional weigh hopper, batch cycle time is reduced by weighing RAP independently and in conjunction with the asphalt and aggregates. The advantage, of course, is that during long production runs of recycled pavement, an increase in production rate per hour can be achieved with the slightly shorter batch cycle time.

3.2 Recycling milled RAP with Hot Mix Asphalt technologies (40% RAP)

In this solution, the proportion of binder that is contributed by milling is very important and is must be considered when calculating the dosage and characteristics of the new binder. The final binder resulting from the blend between the old binder of the milled material and the new binder must meet the same characteristics as for conventional hot mix asphalt. With this purpose, the new binder used is a special product developed by REPSOL with suitable
Table 2. Summary of RAP and binder laboratory test results

<table>
<thead>
<tr>
<th>TEST</th>
<th>Aged RAP bitumen</th>
<th>Rejuvenating special bitumen</th>
<th>Bitumen al 40% RAP HMA</th>
<th>STANDARD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Penetration 25ºC (1/10mm)</td>
<td>14</td>
<td>126</td>
<td>49</td>
<td>UNE-EN 1426</td>
</tr>
<tr>
<td>Softening point (ºC)</td>
<td>69,0</td>
<td>43,4</td>
<td>53,6</td>
<td>UNE-EN 1427</td>
</tr>
<tr>
<td>Penetration Index</td>
<td>0,01</td>
<td>-0,58</td>
<td>-0,34</td>
<td>UNE-EN 12591</td>
</tr>
<tr>
<td>FRAAS breaking Point</td>
<td>+1</td>
<td>-22</td>
<td>-12</td>
<td>UNE-EN 12593</td>
</tr>
<tr>
<td>Ductility</td>
<td>9</td>
<td>&gt;100</td>
<td>73</td>
<td>UNE-EN 13398</td>
</tr>
</tbody>
</table>

This solution has more complexity due to the old bitumen contribution. It is defined on Article nº22 of Spanish specifications (PG-4 from the Ministry of Public Works) as a technique consisting in the use of material resulting from the disintegrated layers of old bituminous pavement mix (by milling or demolition-crushing) in the manufacture of hot mix asphalt. Article 22 also specifies that a recycled bituminous mixture shall contain a mass proportion of the bituminous material to be recycled between ten (10%) and fifty percent (50%) of the total mass of the mixture. In the case of the Arlanzón highway, the RAP percentage added to the mix has been 40%. With this type of mixture 26,480 ton of RAP has been consumed. Same tests as conventional mixture must fulfill for the mixture design are carried out for the recycled mix. The properties of this mix are show in table 3.

Table 3. Summary of 40% RAP asphalt mix laboratory test results

<table>
<thead>
<tr>
<th>HOT MIX ASPHALT type</th>
<th>AC 22 BASE 35-50 G (40% RAP)</th>
</tr>
</thead>
<tbody>
<tr>
<td>% bitumen (weigh) / mix</td>
<td>4,12</td>
</tr>
<tr>
<td>% bitumen weigh) / aggregates</td>
<td>4,30</td>
</tr>
<tr>
<td>% rejuvenating bitumen (weigh) / mix</td>
<td>2,53</td>
</tr>
<tr>
<td>% rejuvenating bitumen weigh) / aggregates</td>
<td>2,59</td>
</tr>
<tr>
<td>Density (gr/cm³)</td>
<td>2,347</td>
</tr>
<tr>
<td>Stability (kN)</td>
<td>1450</td>
</tr>
<tr>
<td>Flow (mm)</td>
<td>2,68</td>
</tr>
<tr>
<td>% VMA</td>
<td>14,76</td>
</tr>
<tr>
<td>% VTM</td>
<td>5,36</td>
</tr>
<tr>
<td>maximum Density (gr/cm³) UNE-EN 12697-6</td>
<td>2,480</td>
</tr>
<tr>
<td>% ITSR</td>
<td>85,6</td>
</tr>
<tr>
<td>Wheel tracking UNE-EN 12697-22</td>
<td></td>
</tr>
<tr>
<td>Slope (mm/1000 cycles)</td>
<td>0,070</td>
</tr>
<tr>
<td>Mean Depth (%)</td>
<td>2,8</td>
</tr>
</tbody>
</table>

This solution has been applied into the highway as thickness of 10 cm thickness base-layer into the prescribed specifications 032 section:

(5 cm wearing course + 5 binder course + 10cm base course) HOT MIX ASPHALT
+ 25 cm CEMENT BOUNDED AGREGGATES
+ 20 cm CEMENT STABILIZED SOIL

In this way, Spanish specifications are met, as Rehabilitation Guidelines 6.3 IC only allows the use of this recycling technique for high traffic category road with at least an upper 10 cm thickness of bituminous mixture.

The manufacturing process at Sacyr Circular Recycling Asphalt Plant is constituted by (figure 4):

- Disintegration of milling in a specific mill
- Classification in two different fractions
- Continuous weight measurement of the manufacture and consumption of each fraction
- Introducing RAP fractions in different access (selective heating)
- Bucket elevation for RAP and standard elevation and screening for virgin aggregates
- Introduce RAP, virgin aggregates, bitumen and filler at the pug-mill for batch mixing
Figure 4. Plant layout for 40% RAP mixes

The technique incorporates a separate convective heat transfer dryer for the RAP, which results in higher percentages than any other single drum batch plan. Recycled mixes with 50% RAP content are typical.

Introducing RAP fractions in two separated access allows selective heating been produced. Each fraction needs heating as a function of the aggregate size and bitumen content. (Less for fine aggregates). RAP materials are weighed as a separate ingredient, and then conveyed to the pug-mill for production of a recycled mix formula.

The RAP-dryer is a parallel-flow design, and the gasses contain steam and hydrocarbons from the convective heat transfer of the RAP. Normally they are exhausted to the primary aggregate dryer where hydrocarbons are destroyed in the combustion area of that dryer. Therefore the percentages of RAP are primarily limited by the capability of the burner on the virgin aggregate dryer to accept the steam and hydrocarbon laden gasses of the RAP dryer. But Sacyr Circular Recycling Asphalt Plant, the gasses from RAP-drier-Drum burn are exhausted backwards to the oxygen entrance. With this technology, the oxygen at the flame is reduce and the hydrocarbons have lower productions rate.

3.3. Recycling milled RAP with Half Warm (under 100°C) technologies, experimental section (100% RAP)

This innovative solution is a development of Sacyr Construcción, which consists on a new mixture that allows incorporating very high rates of recycled material, by means of half-warm temperature technology. It is possible to achieve a 100% rate, complete RAP recycling. The resulting mixes, also present high levels of mechanical and functional characteristics that allow the use in replacement layers, without the limitations inherent to cold recycling techniques.

At Arlanzón highway, recycled Half Warm 100%-RAP has been used for the first time. Manufacturing technology allows heating the bituminous material directly but preserving the quality and integrity of the binder.

For this solution, binder is provided as emulsion. Achieving 100% RAP, it is essential to analyze the characteristics of aged bitumen from milled material; properties of the new emulsion are defined by the nature of the old binder. Because this technique is a wide world novelty and due to the lack of references, a new design methodology has been developed ad-hoc by Sacyr, for the 100% RAP recycling mixture.

This mixture has been extended as an intermediate layer in an experimental section located on a side road, with a length of 1,300 m and with lower traffic category compared with the center lanes.

There are not any specifications from Spanish Ministry that about half warm recycled asphalt. So, in this case, taking a conservative position, the specification for cold recycling has been meted. (Section 7.4. Of Standard 6.3-IC although its properties are in a lower level from half warm mix.)

The experimental section is built with 5 cm milling of pavement on service, replaced with half warm 100% RAP mix, and the upper layer conventional concrete wearing dense asphalt course, which fulfill the minimum cover indicated in Spanish specifications for cold recycling included in the Rehabilitation Guidelines 6.3-IC.
Figure 5. Plant layout for 100% RAP mixes

The manufacturing process at Sacyr Circular Recycling Asphalt Plant is constituted by (figure 5):

- Disintegration of milling in a specific mill
- Classification in two different fractions
- Continuous weight measurement of the manufacture and consumption of each fraction
- Introducing RAP fractions in different access (selective heating)
- Bucket elevation for RAP and standard elevation and screening for virgin aggregates
- Introduce RAP, virgin aggregates, adding emulsion at the pug-mill for batch mixing

Because this technique is a wide world novelty and due to the lack of references, a new design methodology has been developed ad-hoc by Sacyr, for the 100% RAP recycling mixture. For more precise explanation, asphalt mixtures between 70ºC-80ºC and under boiling water temperature 100C, are classified as HALF WARM ASPHALT MIXES, (HWAM)

For the study of this new methodology, it is important to analyze the behavior of bitumen under 100º Celsius. At the temperature, the bitumen from RAP aggregates is over its softening point. Although it has not reach the melting point yet. So, the RAP particles are not having a solid behavior, as they have when using emulsion or hot liquid bitumen. Plastic deformation is taken place in old bitumen which absorbs part of the compaction energy applied. The design methodology must consider this singularity. So, the samples manufactured at the laboratory need to get as closer to the real extended pavement as possible. This is the reason why contrasting the laboratory and industrial compaction it is indispensable.

4. CONCLUSIONS AND FUTURE DIRECTIONS

It’s almost a decade that Sacyr started this long way with its Circular Green Asphalt Plant, convinced about the opportunity that RAP materials represent a chance to improve existing technology, to implement sustainable development principles and to reuse a technically and economically valuable material. But Arlanzon Project described in this paper it’s not a full stop but a semi-colon. Some objectives have been adapted to actual results, others have been fine-tuned and, furthermore, new paths have appeared that in the next years will allow Sacyr continue offering the most outstanding and the most recent emerging technologies in the pavement field. These steps are only the beginning point of new activities in these fields that, through technology transfer, Sacyr will be willing to implement in the international markets where it is working.

Administrations and clients have shown a great interest on these technologies. It is necessary their continuous support for implementing these techniques in designs, developing R&D programs on sustainability and promoting
politic strategies based on “polluter pays” principle, favoring those initiatives proposing sustainable materials, technologies and designs that minimize waste, natural resources consumption and CO2 emissions.

5. REFERENCES

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**PAPER TITLE**

*Rigid pavements, the use of Rubberized Tyre Waste material for improve sustainability and reduce environmental impact*

**TRACK**

Sustainable Pavement Design & Management

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**KEYWORDS:**
Rubber Tyre Waste, Concrete, Pavements, Sustainability

**ABSTRACT:**

It is an extended and accepted idea that Rigid pavements have a higher construction cost than flexible pavements and that its maintenance cost is lower. Nowadays public administrations and concessionaires are analyzing to use them and many times they are rejected due to the future pathologies.

In general terms, the durability of the rigid pavements it is affected by the presence of cracks and for losing sealant material in the joints. This situation, with the accumulation of water and the effect of the traffic loads, produces fines migration and scouring that affect the foundation of the slab and causes its failure.

The submitted study analyses how the addition of rubberized tire waste (RTW) material improves the durability of the concrete delaying the failure by fatigue of the slab.

Determine the composition of the concrete to improve its specifications in terms of flexibility by the addition of RTW and analyze the mechanical characteristics of modified rigid pavements has been considered the technical goals in this study.

In terms of the general goals the study searches for new uses of RTW, safe natural resources, reduce the traffic noise, increase the adherence between vehicle and road and to extend the life-span of the road and reduce the maintenance costs.

The results of the present study show how the concrete mechanical characteristics are affected using different RTW additions percentages and how using RTW material in concrete can be a good solution to recycle an environmentally dangerous material.
Rigid pavements, the use of Rubberized Tyre Waste material for improve sustainability and reduce environmental impact

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1. INTRODUCTION: Current conditions and Objectives of this study

World population is growing day by day; according to the UN the current world population in 2017 has reached 7,500 million and the prospects for 2030 are that it will reach 8,551 million, around 14% increase [1]. The world population needs and wastes will increase drastically in the following years. Nowadays the number of registered passenger’s cars reached 500 million expecting for 2030 to triple the number [2]. Considering this scenario and the fact that those cars will continue to use rubber tyres the United States Tire Manufacturers Association (USTMA) estimated in 2015 that the net scrap tyre generation will reach more than 4 million tonnes per year in United States [3]; while the European Tyre & Rubber Manufacturer’s Association (ETRMA) estimated a 2% growth per year in tyre production in Europe reaching in 2015 more than 4.9 Million tonnes.

Without a proper management tyre scrap material suppose a threat to public health and safety. Stockpile tyres are proven to be a health hazard as tyres are a bulky material full of voids that can hold water for long term and will provide an ideal habitat for the mosquitoes and vermin. Mosquitoes are attracted to the rainwater that builds up in the wells of the tyres, and to the warm, dark environment; a single tyre can nurture hundreds of larvae. Diseases associated with mosquitoes include dengue fever malaria among other viruses.

Considering the safety there is a high risk in case that stockpiled tyres catch fire, the intense radiant heat can cause damage to neighbouring properties and inhibit fire-fighting efforts, and the incomplete combustion of tyres can cause a health risk through the inhalation of particulates. Tyre fires are very difficult to extinguish and are dangerous to firemen. Environmental impacts to soil, watercourses and storm water are likely, and clean-up costs can be extremely high as some oils are spilled out due to the disintegration of the tyres by the fire. [4]

The scrap tyre material can be seen in both ways as a waste and as a new resource and opportunity; several solutions for this material have proved that there is a second life by using this material as ground rubber, land disposed, and electric arc furnace and as an aggregate for pavement bituminous and concrete pavement. [3]

The concrete pavements have recognized advantages that prove them as one of the most convenient solutions not only from the technical point of view, but also the economic and environmental.

Concrete pavements have many advantages, they have a long useful life in different climates and conditions of services have also low maintenance costs; a simple equipment it’s required to be built with low energy consumption and very good characteristics in terms of non-slip surfaces and resistance to aggressive agents and others.

European Concrete Paving Association (EUPAVE) states that concrete pavements are a viable solution to achieve sustainable construction and meet the basic requirements of protection of the environment, economy and society [5].

This paper is focused in the use of rubberized tyre waste material as an aggregate for concrete rigid pavement as other use for this material looking for the size and percentage to be used in order to reduce the affection in terms of resistance of the concrete after adding the material.

2. VARIATIONS IN THE CHARACTERISTICS OF THE CONCRETE AFTER ADDING RUBBERIZED TYRE MATERIAL

Since the last century up to now many studies have been made in order to analyse the variations in the concrete specs when some of the aggregates are substituted by rubber. All of them have in common that the predominant sizes of rubber used can be classified as follows: “coarse aggregate” particles larger than 4.75 mm, “fine aggregate” particles between 4.75 mm and 75μm as Cairns stated [6] and “crumb rubber” that it is a thin material of approximately 4.75 mm as per Khaloo Ali R. studies [7].
In terms of workability the shape and size of the material becomes relevant for the addition in the concrete, Khaloo Ali R. [7], workability it is reduced the bigger the size and amount of rubber added, it is not acceptable to include a rubber percentage of 40% as it makes the concrete unworkable, in other hand adding thin rubber particles improves the workability comparing it with an ordinary concrete.

Considering compaction [8] including rubber in the concrete reduces its homogeneity. The amount of rubber and size affects the density and the absorption of the concrete. Rubber reduces the density as per the lowest weight of rubber comparing it to the ordinary aggregates [9] and it increments the permeability of water in the mix and in case that rubber replaces the course aggregate it increases the absorption of water.

One of the main reasons of the uses of concrete is its resistance to different loads and efforts, Concrete has a very good response against compression and it is reinforced with steel as it is not as efficient in terms of traction loads. Adding rubber to the concrete, as a flexible material it is, will definitely affect the mechanical characteristics of the concrete for that reason the kind, shape and size of the rubber acquires relevance as the better interaction between both materials will result a product with small differences with the ordinary concrete and also in some cases it will enhance some other characteristics of it.

It is commonly accepted and probed that rubber reduces the resistance against compression [9][10][11], in order to reduce the impact of the addition of rubber to the concrete some studies searched a pre-treatment of the rubber in order to improve the interaction between rubber and concrete reducing the air between the particles [6][11] but as the objective is to reduce the environmental impact of the rubber and make it an attractive alternative to substitute a portion of the aggregates, the inclusion of additives to improve the interaction will need a feasibility study in both ways economical en environmental. Some researches in this regard conclude that limiting the percentage of rubber will allow the concrete to have an acceptable resistance that make it appropriate to be used in non-primary structural applications [6] such as pavements.

Modulus of elasticity, resistance to bending stresses and traction are also affected by the inclusion of rubberized tyres material and percentage and size of it [12], shrinkage cracking is also reduced as rubber is more flexible than concrete and for that reason it improves this characteristic by using it in the mix [9], the resulting product is also more malleable than the ordinary concrete [9] making it more durable against cracking.

All of these variations in the concrete characteristics make the concrete with rubberized tyre material a non-suitable material for primary structures but as it improves the resistance against cracking and makes it better in terms of noise [13] it is a good alternative to conventional concrete pavements.

3. METHODOLOGY

This study analyses how the concrete is affected in terms of resistance and life-span using different percentages of rubber (10%, 20% and 30%) and different sizes (1-4 mm, 10 mm and 16 mm).

As the purpose is to determine the resistance against transversal cracking, the proposed test is to apply cyclical loads in order to find the fatigue of the material.

The first step is to design the mix, for that purpose, this study applies the Spanish concrete regulation [14] to select the components for the concrete and the main properties of it that are summarized below:

<table>
<thead>
<tr>
<th>Material</th>
<th>Description</th>
<th>Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>Portland CEM I 42.5-R</td>
<td>3.03</td>
</tr>
<tr>
<td>Sand</td>
<td>Siliceous sand, size 4 mm. Sand equivalent &gt; 90%</td>
<td>2.63</td>
</tr>
<tr>
<td></td>
<td>Siliceous gravel, maximum size 32 mm. Absorption</td>
<td></td>
</tr>
<tr>
<td>Large aggregates</td>
<td>0.54%, Los Angeles coefficient 29%, Shape</td>
<td>2.67</td>
</tr>
<tr>
<td></td>
<td>coefficient 0.24</td>
<td></td>
</tr>
<tr>
<td>Plasticizer</td>
<td>SIKA®, percentage 0.1%</td>
<td>1.08</td>
</tr>
</tbody>
</table>

The following Table 2. describes the granulometry of the material.
Table 2. Granulometry of the Rubber material

<table>
<thead>
<tr>
<th>Granulometry</th>
<th>Density (g/cm³)</th>
<th>Coef. Absorption (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 – 4 mm.</td>
<td>1.132</td>
<td>2.24</td>
</tr>
<tr>
<td>10 mm.</td>
<td>1.128</td>
<td>1.47</td>
</tr>
<tr>
<td>16 mm.</td>
<td>1.134</td>
<td>1.26</td>
</tr>
</tbody>
</table>

The tests were designed in order to measure the deformation due to cyclic load stresses to determine the resistance of the samples measuring the deformation. As it is well known that the breaking point for this kind of stresses are lower than the ones obtained applying a constant stress. The fatigue of the material occurs if the number of loads and unloads are high.

The dimensions of the samples were 100x100x400 mm and the load test were done taking into consideration the Spanish standards [15][16][17], the samples were tested after 28 curing days having three samples for each mix. Table 3 shows the specifications for each mix tested.

Table 3. Mixes specifications

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Cement [kg/m³]</th>
<th>Fine aggregate [kg/m³]</th>
<th>Coarse aggregate [kg/m³]</th>
<th>Water [kg/m³]</th>
<th>Plasticizer [%]*</th>
<th>Rubber grading</th>
<th>Rubber content [kg]</th>
<th>Rubber content [%]*</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC0</td>
<td>397.05</td>
<td>623.2</td>
<td>1,153.44</td>
<td>200.00</td>
<td>0.1</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>RC1</td>
<td>397.05</td>
<td>623.2</td>
<td>1,153.44</td>
<td>200.00</td>
<td>0.1</td>
<td>1-4 mm</td>
<td>14.81</td>
<td>10%</td>
</tr>
<tr>
<td>RC2</td>
<td>397.05</td>
<td>623.2</td>
<td>1,153.44</td>
<td>200.00</td>
<td>0.1</td>
<td>1-4 mm</td>
<td>29.62</td>
<td>20%</td>
</tr>
<tr>
<td>RC3</td>
<td>397.05</td>
<td>623.2</td>
<td>1,153.44</td>
<td>200.00</td>
<td>0.1</td>
<td>1-4 mm</td>
<td>44.42</td>
<td>30%</td>
</tr>
<tr>
<td>RC4</td>
<td>397.05</td>
<td>623.2</td>
<td>1,153.44</td>
<td>200.00</td>
<td>0.1</td>
<td>10 mm</td>
<td>14.81</td>
<td>10%</td>
</tr>
<tr>
<td>RC5</td>
<td>397.05</td>
<td>623.2</td>
<td>1,153.44</td>
<td>200.00</td>
<td>0.1</td>
<td>10 mm</td>
<td>29.62</td>
<td>20%</td>
</tr>
<tr>
<td>RC6</td>
<td>397.05</td>
<td>623.2</td>
<td>1,153.44</td>
<td>200.00</td>
<td>0.1</td>
<td>10 mm</td>
<td>44.42</td>
<td>30%</td>
</tr>
<tr>
<td>RC7</td>
<td>397.05</td>
<td>623.2</td>
<td>1,153.44</td>
<td>200.00</td>
<td>0.1</td>
<td>16 mm</td>
<td>14.81</td>
<td>10%</td>
</tr>
<tr>
<td>RC8</td>
<td>397.05</td>
<td>623.2</td>
<td>1,153.44</td>
<td>200.00</td>
<td>0.1</td>
<td>16 mm</td>
<td>29.62</td>
<td>20%</td>
</tr>
<tr>
<td>RC9</td>
<td>397.05</td>
<td>623.2</td>
<td>1,153.44</td>
<td>200.00</td>
<td>0.1</td>
<td>16 mm</td>
<td>44.42</td>
<td>30%</td>
</tr>
</tbody>
</table>

* % with respect the cement volume

The samples are subjected to flexural stress that it is characterized by the following values: maximum value (σ_{max}) and minimum value (σ_{min}) of the stresses, the difference between them (Δσ_F); average value (σ_{avg}).

The fatigue test it is commonly defined as the one that applies a load during n cycles (n= i…j) with a maximum and minimum value in each cycle obtaining the value of Δσ_F that determines the fatigue limit. A sample suffers fatigue breaking risk in case that:

σ_{max} - σ_{min} > Δσ_F

(1)

In the designed test the sample suffers a pre-load of 70% of breaking load with an increment of 5%, preloaded value it is reached in three stages in order to avoid any problem in the sample. The frequency of the load was 2.5 cycles/sec. until the sample breakage.

This test also incorporated a device to measure deformation suffered by the samples due to the cycle load stress, this device used strips to measure the deformation glued to the top and bottom of the samples. All the deformations are collected by computer. Fig. 1. Shows an equivalent equipment to the one used in this research.
5. RESULTS

Compressive stress results:
Considering a percentage of rubber in the material of 10%, the three different sizes of the rubber obtain similar results with a moderate reduction in the resistance (between 0.79 and 0.85). Considering the smallest size proportion (1-4 mm) the loss in resistance for the higher percentages is similar (0.7590 and 0.7580). If we focus in the intermediate granulometry (10 mm) when the percentage of rubber is increased from 10% to 20% the loss in the resistance is almost inexistent (0.8034 to 0.7959) but this loss grows with the percentage of 30% reducing it to 0.6816. In the larger granulometry (16 mm) the percentage of 10% of rubber shows the lowest loss in terms of resistance, however when this percentage of rubber grows it is the portion that suffers the highest loss in the resistance, 0.7325 (20%) and 0.6822 (30%)

Un-direct tensile stress results:
In the lower granulometry (1-4 mm) the resistance loss is minimum for percentages of 10% of rubber (0.9245) and 20% of rubber (0.9157) being drastically increased when the percentage grows to 30% (0.8082). Considering the portion of 10 mm rubber with a percentage of 10% the loss in the resistance shows acceptable values but by increasing the percentage of rubber to 20% (0.8311) and 30% (0.8012) the loss reaches higher values. The last granulometry of 16 mm shows a behaviour similar to the compressive stress results, in the percentage of 10% the loss is 0.9677, considering a 20% rubber material the loss reaches 0.8441 with a moderate value but for the percentage of 30% the loss reaches 0.6625 being the highest loss of all the test.

Flexural stress results:
The analysis of the results according to this stress shows that the sample with lower granulometry (1-4 mm) and lower percentage of rubber (10%) the result improves the conventional concrete (1.0837) for the rest of the granulometries the resistance suffer losses. The granulometry of 10 mm. shows lower loss in the resistance, from 0.9883 (10%) to 0.9416 (20%). If the percentage reaches 30% the smallest granulometry shows a lower loss (0.9066). As it happened in the
previous stresses the biggest size of the material with the lowest percentage shows a small loss (0.9650) that grows drastically with the increase of the percentage of rubber 0.9144 (20%) and 0.8132 (30%)

The following table (Table 4.) collects the resistance values to the stresses of compression, un-direct tensile stress and flexural stress obtained for all the samples and comparing it to a conventional concrete sample.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Compressive Stress (N/mm²)</th>
<th>Strength Reduction</th>
<th>un-direct tensile stress (N/mm²)</th>
<th>Strength Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC0</td>
<td>38.51</td>
<td>1.0000</td>
<td>33.1</td>
<td>1.0000</td>
</tr>
<tr>
<td>RC1</td>
<td>30.55</td>
<td>0.7933</td>
<td>30.6</td>
<td>0.9245</td>
</tr>
<tr>
<td>RC2</td>
<td>29.23</td>
<td>0.7590</td>
<td>30.31</td>
<td>0.9157</td>
</tr>
<tr>
<td>RC3</td>
<td>29.19</td>
<td>0.7580</td>
<td>26.75</td>
<td>0.8082</td>
</tr>
<tr>
<td>RC4</td>
<td>30.94</td>
<td>0.8034</td>
<td>30.31</td>
<td>0.9157</td>
</tr>
<tr>
<td>RC5</td>
<td>30.65</td>
<td>0.7959</td>
<td>27.51</td>
<td>0.8311</td>
</tr>
<tr>
<td>RC6</td>
<td>26.25</td>
<td>0.6816</td>
<td>26.52</td>
<td>0.8012</td>
</tr>
<tr>
<td>RC7</td>
<td>32.93</td>
<td>0.8551</td>
<td>32.03</td>
<td>0.9677</td>
</tr>
<tr>
<td>RC8</td>
<td>28.21</td>
<td>0.7325</td>
<td>27.94</td>
<td>0.8441</td>
</tr>
<tr>
<td>RC9</td>
<td>26.27</td>
<td>0.6822</td>
<td>21.93</td>
<td>0.6625</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Flexural stress</th>
<th>Resistance (N/mm²)</th>
<th>Strength Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC0</td>
<td>5.14</td>
<td>1.0000</td>
</tr>
<tr>
<td>RC1</td>
<td>5.57</td>
<td>1.0837</td>
</tr>
<tr>
<td>RC2</td>
<td>4.76</td>
<td>0.9261</td>
</tr>
<tr>
<td>RC3</td>
<td>4.66</td>
<td>0.9066</td>
</tr>
<tr>
<td>RC4</td>
<td>5.08</td>
<td>0.9883</td>
</tr>
<tr>
<td>RC5</td>
<td>4.84</td>
<td>0.9416</td>
</tr>
<tr>
<td>RC6</td>
<td>3.95</td>
<td>0.7685</td>
</tr>
<tr>
<td>RC7</td>
<td>4.96</td>
<td>0.9650</td>
</tr>
<tr>
<td>RC8</td>
<td>4.7</td>
<td>0.9144</td>
</tr>
<tr>
<td>RC9</td>
<td>4.18</td>
<td>0.8132</td>
</tr>
</tbody>
</table>

**Elasticity**

The results for the elasticity of the samples comparing it to the conventional concrete shows that independently of the size of the rubber particles there is a considerable reduction in the module as it can be seen in the figure 2.

**Fatigue Resistance**

The results of the fatigue shown that the samples with 1-4 mm and 20% of rubber (RC2), 10 mm and 10% rubber (RC4), 10 mm and 20% rubber (RC5) and 16mm and 10% Rubber (RC7) increased its deformation with only a small reduction (<0.9) in resistance considering the plain concrete
After analysing these samples against load cycles it could be seen that the RC4 (10mm and 10%) can bear with the highest number of load cycles (figure 4.) against the number of load cycles that the conventional concrete could bear (7075 load cycles)

![Figure 3. Deformation vs Load Cycles (RC4)](image)

6. DISCUSSION

From the results it can be seen that the optimal percentage of added rubber is related with the size of the added material. Elasticity module is reduced drastically by the addition of rubber, showing that the new material is more flexible than the conventional concrete; its behaviour in terms of breakage of the samples can be related to an improvement in the plastic behaviour when the percentage of rubber in the mix is high. The addition of rubber improves the cohesion of the materials delaying fragile failures; all of these new characteristics make it suitable to be used in rigid pavements as the concrete pavements are massively exposed to cyclical loads.

As it could be seen in the results, for some percentages and rubber sizes the increase in the deformation against a small loss of the resistance, it is possible to adjust and improve the contact between rubber and cement so that the deformation can be improved and also the number of cycles of loads to be borne.

In all the sizes of rubbers when the percentage of rubber is high (30%) the behaviour of the samples show a descent in the resistance and deformation values due to the contact between materials with very different deformation values.

7. CONCLUSIONS

Adding rubber to the concrete improves deformation and increases resistance against fatigue loading (Table 3), those characteristics of new durability against fatigue and flexibility makes the rubber concrete a good material to be used against the commonly known failures in concrete pavements.

Considering the size and percentage the samples with 10% of rubber and granulometry of 10 mm shown that with a constant deformation can bear with the highest number of load cycles.(Figure 8.),

Percentages of rubber addition over 20% can be rejected due to the decrease in the mechanical characteristics of the concrete.

Rubberized tyre material to be used as an aggregate in concrete pavements is an alternative to improve the sustainability, reduction of noise, use of concrete pavements with longer life-span and a reduction in maintenance costs and reduce the environmental impact of this damaging waste as part of the pavement, that can be seen as a chance to reduce the stockpiled material in developing countries.
8. ACKNOWLEDGEMENTS

We would like to thank the Universidad Politécnica de Madrid Lab staff for their effort and enthusiasm in this research and the Ministry of Infrastructure Development of United Arab Emirates for their support.

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[17] Asociación Española de Normalizacion y Certificacion (AENOR), UNE-EN 12390-1. Testing hardened
ABSTRACT:
The United Arab Emirates (UAE) federal government requests that sustainable solutions be explored in the design stage. One option is to use a geogrid in the pavement structure by placing it in the asphalt layer to control reflective cracking. Placing a geogrid in the aggregate layer reduces the pavement structure’s total thickness; it also increases pavement life and reduces cost, material use, and carbon emissions. Geogrids have been used in several roads in Abu Dhabi (Abu Dhabi Municipality), as well as in other Gulf Cooperation Council (GCC) countries (Kuwait, Qatar, and Oman), mainly for roads built on weak soil such as Sabkha.

A geogrid is formed by a regular network of tensile elements with apertures of sufficient size to allow interlocking with surrounding soil, rock, or earth to function primarily as reinforcement. A geogrid may be formed by either stretching and drawing a punched sheet of polymer, by welding together highly oriented discrete bars of polymer, or by weaving together discrete polymer bars into a network that may subsequently be coated if necessary to protect the polymer strips. Geogrids should be manufactured using high-density polyethylene (HDPE), polypropylene (PP), and/or polyester (PET).

The use of geogrids in pavement layers is governed by two criteria: specifications and this acceptance guideline. This paper discusses the specifications and criteria required for the use of geogrids in pavement structure.
1 INTRODUCTION

The United Arab Emirates (UAE) federal government requests that sustainable solutions be explored in the design stage. One option is to use a geogrid in the pavement structure by placing it in the asphalt layer to control reflective cracking. Placing a geogrid in the aggregate layer reduces the pavement structure’s total thickness; it also increases pavement life and reduces cost, material use, and carbon emissions. Geogrids have been used in several roads in Abu Dhabi (Abu Dhabi Municipality), as well as in other Gulf Cooperation Council (GCC) countries (Kuwait, Qatar, and Oman), mainly for roads built on weak soil such as Sabkha.

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2 GEOGRIDS MATERIAL AND USE

Geogrids, are synthetic fabric materials that is used in pavements for separation, reinforcement, filtration, or drainage. They can also be combined with asphalt binders to form a waterproofing membrane. Geogrids use different synthetics and different products, that is why their properties and characteristics vary. Modern pavement designers have limited fundamental modeling of how the inclusion of geogrids impacts pavement design. Some agencies recommend avoiding the inclusion of geogrids as structural elements in pavement designs until the industry better understands their fundamental properties and developed proven standards for their inclusion.

However, some designers use geogrids on a small scale and monitor the pavement performance over time to better understand their impact on pavement designs. Per AASHTO’s Recommended Practice for Geosynthetic Reinforcement of the Aggregate Base Course of Flexible Pavement Structures (AASHTO R 50-09), designers should conduct field tests by including geogrids to reinforce sections of aggregate base layers in pavement structures. Suppliers of geogrids should follow this process to provide sufficient evidence of such materials’ potential adequacy in pavement structures.

Geogrids are used to reinforce asphalt to control reflection cracking as described in Austroads, (2008). The action of geogrid reinforcement in controlling reflection cracking is different to that of a Strain alleviating membrane interlayer (SAMI). SAMI treatments, including geogrid reinforced seals, use bitumen as a waterproofing and strain alleviating membrane layer. Geogrids control strain in the asphalt through the tensile strength of the reinforcing grid.
Some research suggests that geogrids may also be used to reduce rutting in asphalt layers or to improve structural performance to allow a reduced thickness of asphalt. Design criteria for such applications are not well defined and selection is largely based on reports of observed performance provide by suppliers.

To avoid void spaces between the geogrid and underlying surface, the geogrid is generally placed on an asphalt corrective layer or held in place with a sprayed seal. Some fibreglass geogrids are supplied with an adhesive backing to hold them in place while placing asphalt. Sliding and buckling of polyester and polypropylene geogrids during the placing of asphalt can be difficult to control unless the geogrid is held in place by a sprayed seal. Geogrids are normally placed directly under the wearing course. Generally, a minimum covering thickness of 50 to 70 mm is recommended to ensure that the geogrid is firmly held within the asphalt structure.

Tensar (geogrid supplier) provide in their design reports summary of the main benefits of geogrids in pavement structure as per the following Figure 1.

![Figure 1. Geogrids benefits in Pavement structure.](image_url)
geogrid for a certain project and how to obtain the design. Different suppliers will provide different geogrids that vary in properties and most importantly performance. The following sections will discuss geogrids specification and design, then it will be concluded with guideline requirements to enhance the use and acceptance procedure of geogrids in pavement construction.

3 GEOGRIDS SPECIFICATIONS

Most of the geogrid specification focus on material physical properties and strength, such as in Australia (Austroads, 2008), which compared different classes based on elongation and a set of strength tests such as Grab, tear and puncture strength. The target values set for these tests were obtained from AASHTO Standard Specification M288-06 (AASHTO 2006) as shown in Table 1.

Table 1. Geogrids properties (Austroads, 2008)

<table>
<thead>
<tr>
<th>Test</th>
<th>Test method</th>
<th>Geotextile class</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Class 1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&lt; 50%</td>
</tr>
<tr>
<td>Grab strength (N)</td>
<td>AS 2001.2.3.2</td>
<td>700</td>
</tr>
<tr>
<td>Sewn seam strength (N)</td>
<td>AS 3708.6</td>
<td>1260</td>
</tr>
<tr>
<td>Tear strength (N)</td>
<td>AS 3708.3</td>
<td>500</td>
</tr>
<tr>
<td>Puncture strength (N)</td>
<td>AS 3708.4</td>
<td>2750</td>
</tr>
</tbody>
</table>


In addition to the strength requirements other department such as Texas Department of Transportation (TXDOT, 2009) also specified shape thickness and Ultraviolet, as shown in Table 2.

Table 2. Geogrids Requirements (TXDOT, 2009)

<table>
<thead>
<tr>
<th>Property</th>
<th>TEST METHOD</th>
<th>Type I (1.6 typical)</th>
<th>Type II (1.6 typical)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aperture shape</td>
<td>ID Calipered</td>
<td>Triangular</td>
<td>Triangular</td>
</tr>
<tr>
<td>Aperture size (in)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rib thickness (mil)</td>
<td>ASTM D1777</td>
<td>50 typical</td>
<td>60 typical</td>
</tr>
<tr>
<td>Rib Shape</td>
<td>Observation</td>
<td>Rectangular</td>
<td>Rectangular</td>
</tr>
<tr>
<td>Flexural Rigidity (mg-cm)</td>
<td>ASTM D1388</td>
<td>250,000</td>
<td>750,000</td>
</tr>
<tr>
<td>Min Radial Stiffness @ 0.5% strain (lb/ft)^3</td>
<td>ASTM D6637</td>
<td>15,430</td>
<td>20,580</td>
</tr>
<tr>
<td>Junction Strength (Efficiency) (%)</td>
<td>GRI-GG2-87</td>
<td>93 min.</td>
<td>93 min.</td>
</tr>
<tr>
<td>Ultraviolet Stability (%)</td>
<td>ASTM D4355</td>
<td>100</td>
<td>100</td>
</tr>
</tbody>
</table>

1. Triangular aperture geogrid nominal aperture size measured perpendicularly from the center of the "base" (of a transverse rib) to center of the opposite node to characterize height of an equilateral triangle.
2. Flexural rigidity is measured in any rib direction.
3. Radial Stiffness is determined from tensile stiffness measured in any in-plane axis from testing in accordance with ASTM D6637-01.
4. Junction strength (efficiency) for triangular aperture geogrid is measured in accordance with GRI-GG2-87 and GRI-GG1-87 and expressed as a percentage of ultimate rib tensile strength.

However, the most important property that will impact the design is the mechanical property that influence performance of the geogrid in the pavement structure. The mechanical properties are addressed in AASHTO R50-09. AASHTO R50-09 required performance testing conducted for each type of geogrid proposed including woven, welded, extruded, punched and drawn biaxial geogrids, and punched and drawn triaxial geogrids. Different performance tests need to be carried for different subgrade conditions within different soil types. Reinforced sections must be compared to corresponding control sections for each
subgrade condition. This testing must be carried at the site by the Contractor or provided by geogrid manufacturer.

As per AASHTO R50-09, while full scale tests are required, these tests are costly and will consume time to obtain the required data. Accordingly, most of the suppliers will carry these tests in an international recognized facility and will use the results of the testing in developing procedures and correction factors for the design process.

4 PAVEMENT DESIGN USING GEOGRIDS

Conventional pavement designs are carried out as per 1993 AASHTO pavement design guidelines or any other pavement design procedures. Then this conventional design is given to the geogrid suppliers who will analyze the pavement design and provide alternative design using geogrids. The geogrid suppliers provide data sheets as per AASTHO R50-09 requirements and carry their own pavement design tool to obtain the alternative design.

As an example, the following pavement section (Figure 2) was designed for a project in the UAE. The main section of the road that is located on regular existing soil however, the client would like to explore the use of geogrid as a sustainable alternative.

![Figure 2. Conventional Pavement structure.](image)

Two different geogrid suppliers were approached for this project. Each supplier provided different alternative design to the original pavement section as shown in Figure 3 and Figure 4. The difference in pavement section comes from the difference in the geogrid type for each supplier.

![Figure 3. Alternative Pavement structure – Supplier 1](image)
As can be seen from the previous figures, that both pavement designs provided reduction in material used, which leads to reduction in virgin material, carbon emission and cost. The use of geogrid in pavement structure is a sustainable solution to reduce the pavement thickness. However, the design of the reduced section relies on the type, strength, and shape of the geogrid, which differ from one supplier to the other. Suppliers provide the alternative design based on their recommended geogrid type according to their own production properties. This is a major concern especially, if there is several suppliers from different countries, where the quality control is not monitored. Accordingly, guidelines need to be placed in the project specification for selection and acceptance of geogrids to reduce pavement structure layers.

5. GUIDELINES FOR USE OF GEOGRIDS IN PAVEMENT STRUCTURE

Having several geogrid suppliers is good practice however, the data from the supplier and the performance of the geogrid need to be verified and confirmed to avoid major road failure. Based on the above discussion and to use geogrid in the pavement structure, the following guidelines need to be followed to control the used and design of geogrids.

These guidelines are based on Abu Dhabi Municipality (ADM 2014) procedures that was developed for Emirates of Abu Dhabi. The guideline cover the acceptance of the geogrid supplier, the geogrid testing, pavement design procedure, construction requirements and quality control / performance testing after construction.

The following are each guideline in more details.

5.1 Geogrid Supplier
1. Geogrid supplier must be approved by the agency as an acceptable geogrid supplier.
2. Supplier should provide previous experience in the Middle East (preferably UAE).
3. Supplier will support the contractor during construction to confirm method of placing and fixing the geogrid in the pavement layer.
4. The contractor will give a warranty on the pavement for any defects that might be caused by the geogrid.

5.2 Geogrid Testing
1. Geogrid Material properties need to be verified by independent laboratory. A certificate demonstrating compliance with the Specification shall be provided by the Contractor to the Engineer for each pavement geogrid material used, and proposed to be used in the Contract.
2. All test results on which the certificates are based shall not be more than one year old, measured from the date of supply to the site.

3. Results of Full scale accelerated testing should be provided. This test should have been carried out in internationally recognized laboratory to assess the general performance of the geo-girds.

4. Additional trial section - Full Scale accelerated testing is required as described in AASHTO R50-09. This trial test will be carried at the commencement of the project by the contractor using the proposed geogrid to evaluate the performance of this type of geogrid and its suitableness to the project condition. Location and length of the test section will be agreed upon with the Ministry and the engineer.

5. The trial section will be built using the alternative pavement option recommended. The performance of this section should be monitored for short period of time to verify its adequacy.

6. Based on this trial section the final recommendation to use or not to use the geogrid will be made.

5.3 Pavement Design

1. Detailed pavement design calculations and report should be submitted for including geogrid within the pavement structure.

2. The design will be based on the conventional design (without geogrid).

3. Provide design should respect the layers used in the conventional design. The alternative pavement design should not eliminate a complete layer from the conventional pavement structure. It should only result in reducing the thickness of the layers as practical.

4. Geogrid use should be limited to aggregate layers only not in the asphalt layer.

5. Several design options should be explored to obtain the most sustainable cost effective design option.

6. The pavement design should be based specifically for the proposed geogrid not for “Equal Geogrids”.

7. Design steps from agency design manual and acceptance criteria should be followed.

8. Detailed Bill of Quantities and cost estimate will be provided for each option.

9. Pavement design will be carried and provided by the supplier / manufacturer. Then it will be reviewed by the Engineer.

10. Pavement design should be based on 1993 AASHTO pavement design guidelines.

11. Pavement design software can be used after approval from Ministry and Engineer.

12. Life cycle cost analysis will be carried and included in the pavement design report to compare between different proposed options.

5.4 Field Construction

1. All acceptance tests and specification for pavement layers should be satisfied and met with or without geogrid.

2. A construction procedure detailing all work shall be prepared.

3. The proposed construction procedure shall be submitted to the Engineer at least 14 days prior to the commencement of any works related to the placement of the pavement geogrid material.

4. No works related to the placement of pavement geogrid material shall commence until the construction procedures have been approved by the Engineer and the Engineer has given the Contractor permission to proceed.

5. The placement and compaction of the aggregate layer on top of a pavement geogrids in the field can result in installation damage to the geogrid materials. This is typically reflected by a reduction of the tensile strength properties of the geogrids. The amount of installation damage is determined by subjecting the geogrids to a backfill and compaction cycle, exhuming the material, and determining the tensile strength retained within the geogrids. The ultimate tensile strength of the uninstalled product is compared to the ultimate tensile strength of the installed product to derive at the installation damage reduction factor

6. Supplying of geogrid rolls with each roll having adhesive tape fixing bands or printing directly on the material identifying the product name, and its manufacturing style code. The
labelling/printing is preferably at 5 m spacing along the length of the roll of pavement geogrid material. If the pavement geogrid product proposed has difficulties with labelling/printing, the supplier is to propose a method of identification to be considered by the Engineer.

7. Deliver geogrid reinforcement to the site at least 14 days prior to commencement of installation.

8. Store geogrid reinforcement to avoid any damage prior to installation. Do not store the reinforcement directly on the ground or in any manner in which it may be affected by heat. The method of storage must be in accordance with any other recommendations set by the manufacturer.

9. Layer under the geogrid shall be prepared prior to placement, providing a level and uniform ground surface, with appropriate clearing and grubbing performed to accomplish this.

10. The pavement geogrid shall be installed in accordance with the lines and grades shown on the plans and specifications. The geogrid shall be oriented such that the roll length runs parallel to the road direction. Geogrid shall be laid flat and smooth directly on the prepared surface. All wrinkles and folds shall be removed.

11. The Geogrid shall be overlapped a minimum of 300 mm in longitudinal directions, or joined as specified in the project plans or as directed by the Engineer. Soft subgrade installations may require a greater overlap and in some cases, geogrids may be joined using cable ties or other suitable methods to maintain the geogrids location and orientation during fill placement.

12. Prior to placement of the aggregate material, the geogrid shall be inspected by the Engineer, to make sure it is placed in the proper location, and has not been damaged during delivery and placement. Damaged geogrid shall be replaced immediately.

13. Care shall be taken to ensure that geogrid sections do not separate at the overlaps during construction. Road base material shall be placed in lift thickness as shown on the plans. Typically, tracked construction equipment shall not operate directly upon the geogrid. A minimum compacted fill thickness of 200 mm is required prior to operation of tracked vehicles over the geogrid. Any ruts occurring during fill placement shall be immediately filled in with a suitable capping material.

14. A representative sample shall be taken from the roll(s) to be tested in accordance with ASTM D4354. The representative sample shall be no less than four linear metres along the roll for the full production width but not within two metres of the start or end of the roll. The sample will be taken for every 15,000 m2.

6. CONCLUSION

There are several benefits for using geogrid in the pavement structure. All of these benefits are sustainable benefits that will reduce raw material usage, reduce emission /carbon footprint which is an environmental advantage. In addition, there will be cost saving in using an alternative smaller pavement section. However, the problem faced is that geogrids differ by different suppliers due to material type and even within same supplier due to shape and strength.

Accordingly, there is a need to provide guidelines that will control and regulate the use of geogrids in pavement structures and will ensure that performance of the pavement section.

Abu Dhabi Municipality (ADM 2014) provided guidelines that were proven to be good procedure to follow to ensure the geogrid use. This guidelines covered the selection of the geogrid supplier, the properties of the geogrid, the pavement design method using geogrids, the construction procedures for the geogrids and performance of geogrids in a pavement structure.
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Polymer Need and Detection in Asphalt Binders

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Abstract

Polymer Modified Asphalt Binders (PMB) are gaining more attraction in the paving industry. This can be attributed to the inherited limitations of the straight run asphalt (Neat Asphalt Binders NAB) to resist temperature and heavy loading traffic. However, the need for modifying NAB as it involves extra cost and difficulties may not be justified.

On the other hand, in case polymers are added to the asphalt binders, the typical question imposed during the quality control “is it there?” needs to be answered. It is a concern to assure the presence of polymers in the binder; although a typical grading process will assure the results of the modifications; but not the type and content, also it is time consuming and inadequate for quality assurance. Another concern is the possibility that enhancement can be a result of mechanical modification. AASHTO T302 presents a method of identification of polymer content of polymer modified emulsions and asphalt binders that uses Fourier Transform Infrared (FTIR) spectroscopy. Unfortunately, this method is only applicable for SBS modified PMBs. This presentation will summarize an investigation to use FTIR as a quick quality assurance tool to detect the presence of various polymer types and contents in Arabian asphalt binders. Results of the study clearly showed that FTIR is a direct method and a timesaving alternative to assure the presence of polymer type and content. Master calibration curves were developed and a methodology for adoption into quality assurance programs was suggested.
Experimental Studies on Cement-Kiln-Dust Stabilized Soils for Rural Road Pavement Construction in Eastern Nigeria

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1 INTRODUCTION

The construction of rural roads in Nigeria has been a great challenge over the years with the situation giving serious concerns to the stakeholders. Many factors have been adduced to the poor conditions of the rural roads which include lack of fund, lack of political will of the government to tackle it, poor soil materials and construction methodology. Rural roads network form about 65% of the estimated 200,000km total road network in the country (Shagaya 2006) and they form the major access through which the rural dwellers communicate with the urban centres.

The rural areas are the agricultural base for most agricultural products and they service the industries in the cities with the raw materials especially the agricultural-based industries. There have been concerted efforts by government over the years to address the problems of rural roads by providing funds and creating agencies for rural road developments especially the creation of the Directorate of Food, Roads and Rural Infrastructures (DIFFRR) in 1986 during the military regime but these efforts have not been able to yield substantial improvement in the conditions of most rural roads in Nigeria. The roads constructed often experience premature failures making rural roads more difficult than before they were constructed. The reason for the poor performance of rural roads constructed may be due to lack of proper understanding of the road soil materials employed in their construction.

The problem of poor soil materials and the challenges they pose to road construction particularly rural roads have been receiving research attention in the last 30 to 40 years in different parts of Nigeria. There have been studies conducted to evaluate the subgrade soil materials in the country to understand the various soils available in different locations by characterizing their engineering properties and to determine their suitability for economic road pavement design and construction. Some of these studies showed the distribution of low-strength or poor soil materials across the country (Adesunloye 1987; Aitsebaomo 1999, Agbede 1999, Aitsebaomo et al., 2013). Some of these soil materials were found to be highly susceptible to erosion forming gullies and creating geohazards in many parts of the country (Akpokodje et al. 2010) particularly in the eastern parts of Nigeria.

Many rural roads traverse these poor soil terrains and they contribute to the poor state of the rural areas in terms of road infrastructures. The need to incorporate stabilization techniques in the construction of rural roads across the country to improve the performance of these poor soil materials has become obvious from the present performance of rural roads built on these poor soil terrains. The application of stabilization has been practiced for well over 5000 years in different parts of the world (Chmeisse 1992) and the application of soil stabilization is still gaining attention as a means of improving, modifying and upgrading the properties of weak soil materials to meet the requirements of road pavement construction.

Many methods of soil stabilization that have been used are application of cement, lime, bitumen, polymers, etc., although these methods have been found to be very effective but their impacts on the construction costs of roads have not encouraged their use. This situation now led to a search for alternative but equally economic ways of stabilizing soils for road construction particularly the rural roads which do not carry high traffic volume.

The application of industrial wastes to soil in civil engineering construction has recently been receiving attention in both research and field application in many parts of the world particularly in the United States of America, Britain, Europe, India, etc. The idea of utilizing industry wastes in productive construction is a sustainable strategy and is being promoted under Green Development Initiative (GDI) to conserve the environment from their negative impacts (LCS 2002; Lee et al. 2010; Lebo & Schelling, 2001). These
industrial wastes include lime kiln dust, flyash, cement kiln dust, foundry sand, marble dust, quarry dust, etc. Other wastes from agricultural based activities are rice-husk ash, corn-husk ash, baggase ash, etc.

The use of cement kiln dust, a waste generated from cement plants in Nigeria, has not been studied for productive application over the years in Nigeria and the waste is generated continuously and stockpiled creating environment hazards. There has been efforts by some of the cement plants to introduce recycling of the waste into the cement production process to reduce their generation but this effort have been faced with other challenges such as cost of installation and maintenance of the recycling systems resulting in higher energy and general operation costs.

These problems as well as the need to improve the transportation infrastructures in the rural areas in the country form the basis of these research programs currently being conducted. Also, this study is aimed at finding a way of using the cement-kiln-dust produced during the manufacture of cement for civil engineering construction to reduce their negative environmental impact on the society particularly the pollution of air, surface and groundwater resources. These studies are aimed at finding an economic and more durable solutions to rural road pavement problems affecting rural road transportation and mobility in Nigeria, particularly in the eastern Nigeria where rural accessibility have been a serious challenge due to poor soil materials used in constructing them. These rural roads experience rapid deterioration and failure resulting to loss of investments, and thereby aggravate the sufferings of the rural dwellers.

This paper therefore presents some results of laboratory studies on the engineering properties of soil samples and the CKD stabilized soil samples conducted on the eastern Nigerian soil from Amatutu/Agulu, Anambra State, Nigeria where there have been occurrences of soil failures such as landslides, gully erosion and disruption to road infrastructures. And with this contribute to the current knowledge on the search for effective and productive potential use of CKD waste in stabilizing soils for rural earth and flexible road pavement construction.

2.0 REVIEW OF PREVIOUS RESEARCH

There have been intensive studies into the application of industrial wastes that constitute environmental hazards in civil engineering construction in many parts of the world such as India, America, Britain, etc. Several industrial wastes such as cement kiln dust, lime kiln dusts, waste tyres, foundry sand, sludge wastes, etc., are being studied experimentally as well as in field application as potential stabilizers in pavement construction (Button 2003; Parsons & Kneebone 2004; Miller & Azad 2000; Miller et al. 1999; Chandra et al. 2002; Basha et al. 2005) and particularly for rural roads. A brief review of these researches on the use of CKD in stabilizing low-quality soil materials from different parts of the world is presented.

2.1 Soil materials for rural roads

The construction of rural roads in Nigeria has been a serious challenge over the years due to many factors such as poor funding, poor soil materials, poor construction methodology, and so on. The impact of poor soil materials is focused in this review since these affect significantly the design as well as the cost of construction and maintenance of rural roads.

Rural roads in Nigeria traverse different soil terrains of different engineering qualities. The soil materials often encountered vary from good lateritic gravel to poor lateritic clays. Other soil materials found along the rural road routes are clay, shale, sand, silt, organic soils such as peats and black cotton soils. Several poor soil materials in Nigeria commonly referred to as problem soils have been studied with a view of identifying and evaluating their engineering qualities so as to determine what type of road pavements to be built on them or what type of stabilizing treatment to give to them.

(Ackroyd 1970) studied some western Nigerian soils and their qualities in road construction development in the area. (Madu 1976) conducted studies on the geotechnical and engineering properties of some eastern Nigerian laterite soil materials. Figure 1 shows the results of studies conducted on the distribution of problem soils in Nigeria (Adesunloye 1987). (Aitsebaomo 1999) studied some subgrade soils in south eastern Nigeria to determine their properties and their impact on road infrastructures development. The extensive Igunmale shale soil material in Benue state, in the central part of Nigeria was studied by (Agbede 1999) to evaluate their geotechnical and engineering properties for infrastructures development in the area.
(Okeke & Agbasoga 2001) studied some gravel soils in Ihiagwa area in the eastern Nigeria for potential use as construction materials. The characteristics of soils of the eastern Nigeria and their role in soil erosion particularly as it relates to Nanka and Ekwulobia areas in Anambra state has also been studied (Okagbue & Ezechi 1988; Okagbue 1992; Onwuka et al. 2012).

Figure 1. Map of Nigeria showing distribution of problem soils (Adesunloye 1987)

Ugbe (2011) has shown from studies conducted on 152 soil samples from parts of Edo and Delta states in the mid-western and southern Nigeria that the soil contains high fine materials ranging from 14% to 56% and gravel contents ranging from 0 to 6%. (Aitsebaomo et al. 2013) showed that similar results were obtained for subgrade soils from some other parts of mid-western Nigeria and that the major soil materials in the study area contains 52.94% of sandy clay with and 15.44% of clay.

From all these investigations, it is clear that the soil materials based on their geology, geological formation, geotechnical and engineering characterization, have varying engineering qualities ranging from good to poor, and these influence their performance as subgrade, subbase and base materials under road pavement especially rural roads. The use of these poorly rated soil materials require improvement by the application of stabilization techniques for economic and durable performance under traffic loads.

The rural roads are divided into two – paved and unpaved. The paved rural roads are mainly asphalt concrete surfaced or bitumen surface-dressed (flexible) pavement roads. The concrete paved surface is seldom used for rural roads except for some special conditions. The unpaved roads consist of the earth or gravel roads. According to (Shagaya 2006), the earth roads constitute about 65% of the total estimated 200,000km of road networks in Nigeria. The gravel roads are not common except on some quarry roads where they are employed due to the nature of traffic experienced and of course to check dust.

The earth roads form the major access through which rural dwellers transport their goods, mainly agricultural products, to the urban areas for marketing and as well obtain goods and services needed in the rural area. The agricultural products are the second largest of the goods and products from which the country derives its gross domestic product earnings apart from oil. With the present fall in oil price over the world, the country’s attention is now being focused on agricultural products which are cultivated in the rural areas.

The conditions of rural roads are deplorable and thus require urgent attention if the goal of the government to derive maximum benefits is going to be realized. Figures 2(a) and (b) show typical conditions of rural roads built on lateritic clay soils in Amatutu area in the eastern part of Nigeria.

The earth roads in Nigeria are not usually properly engineered and constructed to the required standards and so they are often characterized by flooding, potholing and erosion. The application of stabilization techniques such as the use of industrial wastes appear to be the best cost-effective alternative of developing rural roads in the different parts of the country especially in the eastern part of Nigeria where the soil erosion and landslide which occur in the area due to the nature of the soil existing there create serious problem and thus constitute a great threat to rural road development.
The potential use of low quality soil materials in rural roads construction in many parts of tropical Africa has received tremendous research attention (Cook et al. 2001; Toole and Newill, 1987; Petts et al. 2006) and that marginal or low-strength soils have been used in countries such as Australia, South Africa, Mozambique, Kenya, Bangladesh, etc, for constructing low-volume roads. Some roads in Malawi were also reported to have been constructed using substandard soil materials and have performed satisfactorily under low-volume traffic.

(a) Akpokodje et al. (2010)                                        (b) Failed Earth Road

Figure 2. Typical failed paved and unpaved earth rural roads at amatutu in eastern nigeria

2.2 Stabilization of soils for rural road construction

Stabilization of soils for road pavement construction has been a major technique employed in improving the engineering properties of soil that fail to meet the acceptable criteria of plasticity index, shrinkage, density and strength. The method has been used on many soil terrains during construction of major roads in different parts of the country. The construction of Apapa-Wharf road through Ijora in Lagos was constructed using cement stabilization of fine grained sandy-silty soils (Ackroyd 1970).

A number of researchers have studied stabilization of problem laterite soils in different parts of Nigeria. Osula (1989;1996) studied the use of admixtures such as cement and lime in stabilizing some problem laterites and further used chemical additive such as sodium chloride together with cement to understand the performance of the stabilized laterite (Osula 1993). (Onyelowe 2012) recently reported the results of studies conducted on cement and also bagasse ash stabilization of lateritic soil from Akwuette in eastern Nigeria. However, the application of stabilization for rural roads has not been a usual practice due to its cost implications on the construction of rural roads.

The potentials of CKD in stabilizing soils for rural roads in Nigeria are not yet studied. The studies on the potential productive application of CKD in Nigeria for road construction in Nigeria started sometimes in 2007 with an intensive research program at the University of Lagos (Oduola 2009) and since this work started, some researchers have also been showing keen interest on the subject. Okafor & Egbe (2013) studied the potential of CKD in improving soils in some parts of eastern Nigeria for flexible pavement construction. The potential of CKD in stabilizing problem soils in the eastern Nigeria to enhance the development of rural roads in the area is studied in this paper.

3.0 MATERIALS AND EXPERIMENTAL METHODS

The material location, sampling and experimental procedures adopted in this study are presented in the following sections.

3.1 Geology of the study area and material sampling

The study area is Amatutu/Agulu area of Anambra state in the eastern part of Nigeria, a few kilometers from Akwa, the state capital. The soil in the area is known to be erosive and susceptible to collapse and several areas of the town have been affected by gully erosion and landslides. Figure 3 shows the study area.
The geology of the study area can be described as located within the Anambra basin consisting of Transgressive Lower Tertiary Sediments (Paleocene-Eocene) and partly regressive upper cretaceous sediments (Camparian-Paleocene) (Aitsebaomo 1999). The soil of Amatutu can be described as reddish brown, fine-grained silty sand with little or no clay content. Due to lack of fines or binder content in the soil, the soil appears to lack cohesion. This makes it to be subject to high erosion vulnerability.

The Anambra Formation of Southern Nigeria was reported to have been formed during the Santonian tectonic event that affected the Southern Benue Trough, of which the first lithic fill of the basin is the marine Nkporo Group, and this include the Nkporo Formation, the Owelli sandstone Formation, and the Enugu Shale (Umeji and Nwajide, 2007). According to (Kogbe 1976) cited in (Okeke 1991), the soil of Amatutu/Agulu/Nanka area has been described as “Nanka sand because of its peculiar properties. The soil belongs to Bende-Amaki Formation and the lithology of the soil consists of fine to coarse sandstone with abundant intercalation of calcareous shale and thin shally limestone below and of loss cross bedded white and yellow sandstone with bands of fine grained sandstone and sandy clay above.” Because of the loose or weak bond between the soil particles, it quickly loses strength and gets eroded easily. Figure 4 shows the geological map of Nigeria showing the location of the study area.

The sampling of the soil was done by using digger and shovels to remove the soils from two sampling points. Disturbed samples of the soil were taken at two selected points, placed in polythene bags and transported to the Materials Laboratory of the Department of Civil Engineering, University of Lagos. Cement Kiln Dust (CKD)
used for the stabilization of the soil samples was obtained from Lafarge/West African Portland Cement, Shagamu Plant, Ogun State, Nigeria.

3.2 Experimental procedures

The tests conducted on the soil samples are divided into two. These are engineering tests and chemical/mineralogical tests on the soil samples. The engineering tests conducted on the samples of soils were the particle size sieve analysis, specific gravity, soil classification, Atterberg limits, compaction test, Californian Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS). The experimental procedures adopted for testing the soils without the CKD stabilizer were as specified in BS 1377 (1990) and the procedures for testing the CKD-stabilized soil samples were as specified in BS 1924 (1990).

The chemical analysis of the CKD sample was done using AARL 900 cement Analyzer shown in Figure 5 at the Quality Control Laboratory of the Lafarge/WAPCO Portland Cement Plant Shagamu, Ogun State. The soil samples were stabilized with CKD by increasing the stabilizer content at intervals of 2% by weight. The maximum addition of the CKD is set at 30% but the tests so far conducted are from 0% to 10% CKD addition. The remaining tests which involve increasing the CKD to 30% are still being conducted.

Other tests conducted but not reported here are the mineralogical analysis of the soil samples, CKD and the CKD-stabilized soil samples. These tests include the X-ray Fluorescence using Energy Dispersive Spectrometer (EDS), X-ray Diffraction test and Scanning Electron Microscopy (SEM). These tests were conducted to determine the mineral compounds in the soils as well as the stabilized soil samples so as to understand the reaction and mechanisms that occurred in the samples which affect the engineering properties of stabilized samples. The results of these will be reported when completed.

4. RESULTS AND DISCUSSIONS

The results of the engineering tests conducted using the standard procedures of BS 1377 (1990) and BS 1924 (1990) are presented as follows.

4.1 Results of engineering tests conducted

4.1.1 Particle size analysis, atterberg limits and soil classification

The results of the natural moisture contents conducted on the two soil samples are shown in Table 1 together with the Atterberg limits and soil classification. The values obtained for the natural moisture contents are 9.80% and 13.20% for samples A and B respectively.

The grain size analyses for the two soil samples A and B were carried out and the results of the grain size analysis conducted using the procedure in BS 1377 (1990) are shown in Figure 6 and 7 for the two soil samples.

The results of Atterberg Limits tests conducted on the two soil samples and the addition of cement-kiln-dust (CKD) in the range of 0% to 10% are shown in Table 1. The variation of the plasticity index (PI) index with
percentage CKD added is shown in Figure 8. From the particle size analysis and the Atterberg Limits test results on the unstabilized soil samples, the soil samples A and B were classified based on AASHTO procedures as A-7-6 (9). Based on the Nigeria Highway Manual (NHM, 2013), the soil samples are classified as S5 subgrade.

According to the AASHTO chart used in the classification of the soils, the samples are rated as fair to poor subgrade soils and so are not excellent materials for subgrade soils under road pavement. Garber and Hoel (1999, 2002) also have stated that soils having coarse grain materials are excellent subgrade soil although they have high permeability.

4.1.2 Specific gravity and permeability test results

The results of the specific gravity of the two soil samples A and B measured are shown in Table 2. The results are 2.65 for soil sample A while soil sample B has a value of 2.652. These values fall within 2.6 and 3.4, the range of specific gravity generally reported for lateritic soil materials (Ogunribido 2012). But according to Bowles (1990), the values of specific gravities ranging from 2.52 to 2.66 are reported for inorganic clay soils. This indicates that the soil samples actually fall into the range of lateritic clay.

The specific gravity is an important parameter in determining the clay content of soil samples, the void ratio as well as the soil porosity which are required in determining the drainage properties of soils. The permeability property of the soil samples was measured using falling head permeameter. This property normally indicates the behavior of the soil when subjected to high ground water under road pavement. The results of hydraulic conductivities of the two soil samples are $2.670 \times 10^{-4}$ cm/s and $2.840 \times 10^{-4}$ cm/s for samples A and B respectively as shown in Table 2. From the permeability results, it shows that the soil samples have low permeability and therefore are fine-grained soils likely to be affected by plastic property.

4.1.3 Compaction and california bearing ratio tests

The compaction test carried out in this study is the West African Heavy compaction procedure since the soils will be subjected to vehicular loads. The results of the compaction tests for the unstabilized soil samples and CKD-stabilized soils samples are presented in Table 3. The dry density of the soil samples A and B before stabilization was found to be 1.886 Mg/m$^3$ and 1.90Mg/m$^3$ respectively. The dry density for the CKD stabilized samples of the two soils ranged from 1.864Mg/m$^3$ for soil sample A at 2% CKD addition to 1.666Mg/m$^3$ for the same soil sample at 10% CKD.
Table 1: Natural moisture content, atterberg limits and soil classifications

<table>
<thead>
<tr>
<th>Samples</th>
<th>Natural Moisture Content (%)</th>
<th>Liquid Limit (%)</th>
<th>Plastic Limit (%)</th>
<th>Plasticity Index (PI) (%)</th>
<th>Shrinkage Limit (SL)</th>
<th>Soil Classification (AASHTO)</th>
<th>Subgrade Soil Classification (NHM, 2013)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Natural Moisture Content (%)</td>
<td>Liquid Limit (%)</td>
<td>Plastic Limit (%)</td>
<td>Plasticity Index (PI) (%)</td>
<td>Shrinkage Limit (SL)</td>
<td>Soil Classification (AASHTO)</td>
<td>Subgrade Soil Classification (NHM, 2013)</td>
</tr>
<tr>
<td>A</td>
<td>9.80</td>
<td>45.67</td>
<td>16.60</td>
<td>29.07</td>
<td>8.60</td>
<td>A-7</td>
<td>S5</td>
</tr>
<tr>
<td>B</td>
<td>13.20</td>
<td>46.00</td>
<td>18.10</td>
<td>27.90</td>
<td>8.60</td>
<td>A-7</td>
<td>S5</td>
</tr>
<tr>
<td>A+2% CKD</td>
<td>-</td>
<td>46.40</td>
<td>17.50</td>
<td>28.90</td>
<td>7.37</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>B+2% CKD</td>
<td>-</td>
<td>47.77</td>
<td>17.93</td>
<td>29.83</td>
<td>7.37</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>A+4% CKD</td>
<td>-</td>
<td>47.53</td>
<td>18.13</td>
<td>32.98</td>
<td>7.23</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>B+4% CKD</td>
<td>-</td>
<td>48.30</td>
<td>18.50</td>
<td>29.80</td>
<td>7.43</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>A+6% CKD</td>
<td>-</td>
<td>48.30</td>
<td>18.20</td>
<td>30.10</td>
<td>21.90</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>B+6% CKD</td>
<td>-</td>
<td>48.97</td>
<td>18.80</td>
<td>30.17</td>
<td>7.17</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>A+8% CKD</td>
<td>-</td>
<td>48.87</td>
<td>18.60</td>
<td>30.27</td>
<td>7.60</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>B+8% CKD</td>
<td>-</td>
<td>49.13</td>
<td>19.34</td>
<td>29.77</td>
<td>7.50</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>A+10% CKD</td>
<td>-</td>
<td>49.67</td>
<td>19.3</td>
<td>30.53</td>
<td>7.50</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>B+10% CKD</td>
<td>-</td>
<td>49.23</td>
<td>19.20</td>
<td>30.03</td>
<td>7.37</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
Figure 6. Particle size analysis for soil sample A

Figure 7: Particle size analysis of soil sample B
Figure 8. Plasticity index vs. percent cement-kiln-dust for samples A and B

Table 2. Permeability and specific gravity of soil samples

<table>
<thead>
<tr>
<th>Soil samples</th>
<th>Specific gravity</th>
<th>Permeability (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>2.650</td>
<td>2.67 \times 10^{-4}</td>
</tr>
<tr>
<td>B</td>
<td>2.652</td>
<td>2.84 \times 10^{-4}</td>
</tr>
</tbody>
</table>

However, as the samples were stabilized with CKD, there was a progressive reduction in the dry densities but the optimum moisture contents of the soil samples increased. There are variations of optimum moisture contents with percent increase of CKD and also of dry density with percent increase in CKD respectively (please see Figure 9 and 10).

The results of the Californian Bearing Ratio (CBR) are presented in Table 4 showing the soaked (for 4 days) and unsoaked CBR of the two soil samples with their corresponding CKD-stabilized samples. The soaked CBR of soil sample A and B are 18.0% and 16.33% respectively. The unsoaked CBR of the two samples are 30.0% and 29.33%.

Table 3. Dry density and californian bearing ratio of the soil samples

<table>
<thead>
<tr>
<th>Samples</th>
<th>Optimum moisture content (OMC) (%)</th>
<th>Maximum dry density (MDD) (Mg/m³)</th>
<th>Bulk density (Mg/m³)</th>
<th>California bearing ratio (CBR)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Soaked (%)</td>
</tr>
<tr>
<td>A</td>
<td>13.30</td>
<td>1.886</td>
<td>2.137</td>
<td>18.00</td>
</tr>
<tr>
<td>B</td>
<td>12.10</td>
<td>1.900</td>
<td>2.130</td>
<td>16.33</td>
</tr>
<tr>
<td>A+2% CKD</td>
<td>15.40</td>
<td>1.864</td>
<td>2.151</td>
<td>29.33</td>
</tr>
<tr>
<td>B+2% CKD</td>
<td>13.40</td>
<td>1.880</td>
<td>2.132</td>
<td>34.00</td>
</tr>
<tr>
<td>A+4% CKD</td>
<td>16.30</td>
<td>1.810</td>
<td>2.105</td>
<td>24.33</td>
</tr>
<tr>
<td>B+4% CKD</td>
<td>15.10</td>
<td>1.850</td>
<td>2.130</td>
<td>34.00</td>
</tr>
<tr>
<td>A+6% CKD</td>
<td>17.17</td>
<td>1.764</td>
<td>2.067</td>
<td>20.30</td>
</tr>
<tr>
<td>B+6% CKD</td>
<td>16.20</td>
<td>1.82</td>
<td>2.115</td>
<td>30.33</td>
</tr>
<tr>
<td>A+8% CKD</td>
<td>18.60</td>
<td>1.722</td>
<td>2.042</td>
<td>18.00</td>
</tr>
<tr>
<td>B+8% CKD</td>
<td>17.50</td>
<td>1.750</td>
<td>2.056</td>
<td>29.33</td>
</tr>
<tr>
<td>A+10% CKD</td>
<td>20.30</td>
<td>1.666</td>
<td>2.004</td>
<td>15.67</td>
</tr>
<tr>
<td>B+10% CKD</td>
<td>18.60</td>
<td>1.72</td>
<td>2.040</td>
<td>27.30</td>
</tr>
</tbody>
</table>

From the test results, it was found that the soaked CBR of the samples increased as the percentage increase of the CKD stabilizer. The increase in strength as measured by the CBR values may have been connected with the hydration reaction of calcium oxide (lime) which is the predominant oxide in the CKD in the presence of water.
during the curing of the samples. Figures 11 and 12 show the variations of the soaked and unsoaked CBR properties of the stabilized soil samples A and B with increasing percentage of CKD.

![Figure 9](image-url)  
**Figure 9.** Optimum moisture content vs. percent cement-kiln-dust of samples A and B

![Figure 10](image-url)  
**Figure 10.** Maximum dry density vs. percent cement-kiln-dust of samples A and B

The 4-day soaked CBR values of the unstabilized soil samples A and B are 18.0% and 16.33% respectively. These are found to be less than 30% as required by the Nigerian Highway Specifications (FMWH 1997) for subbase materials under road pavement. However, the soil samples meet the requirement of subgrade materials of not less than 15%. The soil samples when stabilized with 2% to 10% CKD showed impressive increase in soaked CBR especially soil sample B meeting the requirement of subbase materials.

![Figure 11](image-url)  
**Figure 11.** Soaked Californian bearing ratio vs. percent cement-kiln-dust of samples A and B

Based on unsoaked CBR, the strength of the soil did not increase but a gradual decrease in strength was noted as the percent increase in CKD stabilizer. This observation may be due to lack of water for the continued hydration reaction of lime in CKD-soil mixture to continue during the unsoaked curing. However, the soaked samples experienced increase in strength and this is possibly due the hydration and cementation reactions from the lime-rich CKD with the soil samples. The further chemical and mineralogical analyses which are to be conducted at the second stage of these studies will capture the cause of this behavior of the stabilized soil samples.
According to (Lebo & Schelling 2001), the design of rural roads based on unsoaked criteria for the soil materials results in uneconomical design because the rural roads are often subject to low traffic unlike the main urban roads. Therefore it was suggested that the unsoaked CBR can be used to design rural roads.

![Unsoaked CBR vs. Percent Cement-Kiln-Dust](image)

Figure 12. Unsoaked california bearing ratio vs. percent cement-kiln-dust of samples A and B

### 4.1.4 Unconfined compression strength

The results of unconfined compression strength (UCS) tests conducted on the soils and the CKD stabilized soil samples are shown in Table 4. The results show both the uncured and cured (in air) for 7, 14, 21, and 28 days for the two soil samples A and B with their corresponding CKD-stabilized samples. All the CKD-stabilized soil samples show a general increase in strength as the days of curing increased.

The unconfined compressive strength has been used to classify soils into different categories for the purpose of determining their properties and suitability for engineering purposes. Different ranges of soils are known based on some set criteria (Das 2000). For instance, soils having UCS values ranging from 0 to 25kN/m$^2$ are classed as very soft clay soils while those with values ranging from 25-50kN/m$^2$ are classed as soft clay soil. Medium soft clay soils have unconfined compression strength values ranging from 50 – 100kN/m$^2$ while stiff clay soil have values ranging from 100–150kN/m$^2$. Very stiff clay soils have unconfined compression strength values ranging from 200-400kN/m$^2$ while values greater than 400kN/m$^2$ are classed as hard clay soils.

Based on these criteria, soil sample A and B (without stabilizer) have unconfined compression strength value of 40.67kN/m$^2$ and 34.57kN/m$^2$ respectively. This indicates that the soil fall into the soft clay category. But as the soil samples were stabilized with CKD, the strength increased with increase in curing age.

Generally, soil sample A increased from 40.67kN/m$^2$ for uncured condition to 98.60kN/m$^2$ at 28 days of curing at 0% CKD addition. However, the strength of the soil samples reduced as the percentage of cement-kiln-dust are added to the soil samples. Similar observations were noted for sample B. The reduction in strength may be due to the chemical reactions and mineralogical influence in the soil-CKD mixtures but this will be understood when the mineralogical and chemical analyses of the soil and the CKD-stabilized mixtures are carried out during the second stage of these investigations. Figure 13 shows the variation of unconfined strength of the soil with the percent addition of CKD for uncured condition of the stabilized samples.

### 4.2 Chemical analysis of cement-kiln-dust

The results of the chemical analysis of the CKD used in the experiment was conducted using the AARL 900 Cement Analyzer and is presented in Table 5. The purpose of this test is to determine the oxide elemental contents of the CKD stabilizer which will influence the engineering behavior of the CKD-stabilized soil samples. During the second stage of these investigations, the elemental contents and mineralogical analyses of the stabilized soil samples will also be determined.
Table 4. Unconfined compression strength results on soil samples and cement-kiln-dust stabilized soil samples

<table>
<thead>
<tr>
<th>Samples</th>
<th>Uncured (kN/m²)</th>
<th>7 days Curing</th>
<th>14 days</th>
<th>21 days</th>
<th>28 days</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>UCS C&lt;sub&gt;u&lt;/sub&gt;</td>
<td>UCS C&lt;sub&gt;u&lt;/sub&gt;</td>
<td>UCS C&lt;sub&gt;u&lt;/sub&gt;</td>
<td>UCS C&lt;sub&gt;u&lt;/sub&gt;</td>
<td>UCS C&lt;sub&gt;u&lt;/sub&gt;</td>
</tr>
<tr>
<td>A</td>
<td>40.67</td>
<td>20.33</td>
<td>49.89</td>
<td>24.95</td>
<td>58.96</td>
</tr>
<tr>
<td>B</td>
<td>34.57</td>
<td>17.37</td>
<td>46.90</td>
<td>23.45</td>
<td>64.94</td>
</tr>
<tr>
<td>A+2% CKD</td>
<td>29.03</td>
<td>14.57</td>
<td>34.86</td>
<td>17.43</td>
<td>58.71</td>
</tr>
<tr>
<td>B+2% CKD</td>
<td>33.53</td>
<td>16.80</td>
<td>44.90</td>
<td>22.45</td>
<td>62.17</td>
</tr>
<tr>
<td>A+4% CKD</td>
<td>21.23</td>
<td>10.67</td>
<td>32.87</td>
<td>16.44</td>
<td>51.08</td>
</tr>
<tr>
<td>B+4% CKD</td>
<td>28.40</td>
<td>14.23</td>
<td>37.78</td>
<td>18.89</td>
<td>46.78</td>
</tr>
<tr>
<td>A+6% CKD</td>
<td>19.6</td>
<td>9.83</td>
<td>30.75</td>
<td>15.38</td>
<td>41.67</td>
</tr>
<tr>
<td>B+6% CKD</td>
<td>28.57</td>
<td>14.30</td>
<td>51.98</td>
<td>25.99</td>
<td>69.05</td>
</tr>
<tr>
<td>A+8% CKD</td>
<td>22.90</td>
<td>11.47</td>
<td>46.76</td>
<td>23.38</td>
<td>57.86</td>
</tr>
<tr>
<td>B+8% CKD</td>
<td>21.20</td>
<td>10.63</td>
<td>38.97</td>
<td>19.49</td>
<td>63.05</td>
</tr>
<tr>
<td>A+10% CKD</td>
<td>12.50</td>
<td>6.30</td>
<td>26.56</td>
<td>13.28</td>
<td>43.98</td>
</tr>
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<td>12.57</td>
<td>6.37</td>
<td>30.81</td>
<td>15.46</td>
<td>49.89</td>
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</tbody>
</table>
From the results of these elemental oxide contents, it was found that the major elements present in the CKD are silica, aluminium oxide, ferric oxide and calcium oxide. The calcium oxide (lime) was found to be the predominant oxide in CKD with a mean value of 57.65% and lime being a good pozzolanic material can increase the strength properties of the stabilized soils particularly soils with high clay content.

TABLE 5. Results of chemical analysis of cement-kiln-dust

<table>
<thead>
<tr>
<th>Parameters Tested</th>
<th>SiO₂</th>
<th>Al₂O₃</th>
<th>Fe₂O₃</th>
<th>CaO</th>
<th>MgO</th>
<th>SO₃</th>
<th>K₂O</th>
<th>F/CaO</th>
</tr>
</thead>
<tbody>
<tr>
<td>15.760</td>
<td>4.800</td>
<td>2.830</td>
<td>58.250</td>
<td>1.620</td>
<td>1.440</td>
<td>0.970</td>
<td>5.150</td>
<td></td>
</tr>
<tr>
<td>15.840</td>
<td>4.760</td>
<td>2.860</td>
<td>58.170</td>
<td>1.600</td>
<td>1.370</td>
<td>1.000</td>
<td>5.900</td>
<td></td>
</tr>
<tr>
<td>15.250</td>
<td>4.570</td>
<td>2.900</td>
<td>57.530</td>
<td>1.600</td>
<td>1.240</td>
<td>1.030</td>
<td>6.400</td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td>16.098</td>
<td>4.698</td>
<td>2.866</td>
<td>57.650</td>
<td>1.612</td>
<td>1.304</td>
<td>1.006</td>
<td>5.936</td>
</tr>
</tbody>
</table>

5 CONCLUSIONS

From the results of tests conducted on the soil samples obtained from Amatutu/Agulu area of Anambra State in the eastern part of Nigeria, which are presented, analyzed and discussed in the above sections, the following conclusions can be made.

(a) Based on the particle size analysis of the two soil samples and the Atterberg limits, the soil samples were classified according to AASHTO Chart as A-7-6(9). These soil samples were further classified according to the Nigerian Highway Manual (2013) to be S5 subgrade class. These further confirm that the soils are not excellent materials for subgrade or subbase soils; however they meet the criteria for subgrade, and can be employed as base materials for rural roads.

(b) The specific gravity of the soil samples and the permeability test results showed that soil samples have low hydraulic conductivity which means that the soils are fine-grained and so are not excellent materials for subgrade under pavement since excellent subgrade materials must be highly permeable. However if the road crossfalls are constructed to meet the specifications for rural road, the materials can be employed for their construction since the traffic is usually low.

(c) The dry-density which measures the strength based on grain-to-grain contacts in the soil samples and the presence of voids in the soils shows that the density of the soils were fairly high and this can be attributed to the amount of clay material in the soil.

(d) From the Californian Bearing Ratio (CBR) test results, soil samples A and B cannot be used as subbase or base materials under road pavement. However, with stabilization with CKD, the soil material properties increased to meet the requirements for subbase. For rural roads since the traffic are usually low, the use of unsoaked CBR for design of the pavement have been suggested by
(Lebo & Schelling 2001), and so if this specification is applied, then all the soil samples can be used as both subgrade as well as subbase under rural road traffic condition.

(e) The unconfined compressive strength of the soils showed that soil sample A and B belong to very soft clay, however with stabilization, the strength of the soils increased with increasing CKD content and as the air-curing days increased. This increasing trend was observed for the two soil samples studied. This can be attributed to the bonding action of the fine-grained clay sizes present in the soils sample by the hydration reaction of CKD with the soils.

(f) Results of the chemical analysis of CKD show that it is rich in lime and so can provide the needed pozzolanic and hydration reactions with the clay content of the soil samples with a resulting material with lower plasticity index.

(g) These results show that CKD can be productively used in improving low-grade soils to meet the criteria for application in rural road construction in the eastern part of Nigeria where this soil has been posing problem.

(h) Further work is still on-going particularly on more addition of cement kiln dust and on the influence of soil chemical and mineralogy on the properties of the CKD-stabilized soil mixtures.

REFERENCES


Physical Characteristics and Temperature Susceptibility of Polyurethane Modified Bitumen

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KEYWORDS:
polyurethane, bio-binder, modified bitumen, temperature susceptibility, rotational viscosity

ABSTRACT:
Recent research has shown that bio-binders can be used as modifiers or extenders or as an alternative replacement for bituminous binder. This research is conducted to investigate the feasibility of using polyurethane (PU) as a modifier in bitumen used in the paving industry. PU is produced from palm kernel oil-based monoester polyol (PKO-p) and 2,4-diphenylmethane diisocyanate (MDI) through prepolymerization and is added as a modifier to a 80/100 penetration grade bitumen. Investigation was conducted to determine the optimal PU ratio for blending with virgin binder, and the ratios selected in this study are (1:0.6) and (1:0.8). Penetration, softening point, ductility, rotational viscosity and storage stability tests were conducted to determine the effect of adding 5, 10, 15 and 20% PU by weight of binder to 80/100 penetration grade bitumen. The temperature susceptibility of the modified bitumen was determined based on the values of the penetration index (PI) and the penetration viscosity number (PVN). Results indicate that the addition of both 5 and 10% PU produced good compatibility in comparison with virgin bituminous binder. The result for storage stability test shows that both modified bitumen are storage stable blends, which is a very important parameter that must be taken into consideration for modified bitumen. The study finds that increasing the amount of PU added to bitumen results in decreased temperature susceptibility. In conclusion, polyurethane modified bitumen has a promising potential for use in the paving industry.
INTRODUCTION

The price of bitumen has been increasing rapidly since the last several years, and this is likely to result in reduced supply of bitumen in the future. In order to reduce reliance on petroleum bitumen, many studies have been conducted which use biomass as an alternative source of environmentally friendly substitute for bitumen. Biomass has been used mainly as bio-renewable fuel to replace fossil fuel. Over the years, various bio-renewable natural resources, including sugars, triglyceride oils, and proteins have been tested as alternative materials for producing adhesives and binders (Airey & Mohammed 2008). Recent research has shown that bio-binders can be used as modifiers (<10% bitumen replacement), extenders (25% to 75% bitumen replacement), or 100% replacement for bitumen.

Previous research on the use of bio-oils fractions as an extender in original and polymer modified bitumen has concluded that the effect of bio-oils is dependent upon many factors, including the base bitumen itself, the source of the biomass from which the bio-oils were derived, and the percentage of bio-oils blended with the bitumen. Investigations have also shown that, up to 9% bio-oil can be blended with bitumen to achieve significant improvement in the performance grade of the bio-oil modified bitumen (Williams et al. 2009). One research used swine manure-based bio-binder as bitumen modifier and results show that it could enhance bitumen’s low-temperature performance (Fini et al. 2010). Investigation of bitumen’s performance when blended with bio-oil generated from waste wood source showed that it can lower the mixing temperature of asphalt mixtures while at the same time improve the high performance of bitumen (Yang et al. 2013). A preliminary study conducted using waste engine oil with Reclaimed Asphalt Pavement (RAP) found that even though bitumen becomes soft when oil is added, the addition of RAP will stiffened the bitumen (Dedene & You 2011). Another research, which was conducted using waste cooking oil, found that the addition of bio-oil decreases complex modulus and creep stiffness, and increases the phase angle and m-value of asphalt binder. This shows that the addition of bio-oil could reduce deformation resistance and elastic recovery performance of control bitumen and improve stress relaxation property; it could also improve the thermal cracking resistance of the control bitumen (Sun et al. 2016).

In Malaysia, biomass waste is one of the key potential energy sources which could meet the increasing demand for energy. These waste materials typically have little or no economic value and often present disposal problem (Hosseini & Wahid 2014). Biomass is a source of bio-oil, and studies have been focusing on the generation and property characterization of bio-oils. Biomass materials such as microalgae (Chailleux et al. 2011), yard waste (Hill & Jennings 2011), and soy fatty acids (Seidel & Haddock 2012) were used as bio-oil resource. The use of these materials can reduce the amount of bitumen consumption and improve the properties of currently available bitumen. Palm oil-based polyurethane is a renewable source which does not cause permanent depletion of the resource, namely palm kernel oil, which has limited global availability (Chee & Badri 2011). The PU obtained from palm kernel oil has the potential of being used as an extender or a modifier in bitumen to improve the performance grade of a binder. Based on the conclusions of these investigations, the use of bio-oils as a bitumen modifier is very promising.

The objective of this study is to evaluate the feasibility of using PU as a modifier for base bitumen. The effects of PU were investigated through physical testing and were compared with conventional bitumen. Temperature susceptibility was also measured to determine the effects of increasing the percentage of PU on the Penetration Index (PI) and Penetration Viscosity Number (PVN) of unmodified bitumen and modified bitumen. Temperature susceptibility is defined as the rate at which the properties of bitumen changes with temperature (Claudy & Martin, 1998). Higher temperature susceptibility of bitumen will cause many problems prior to and during service.
2 EXPERIMENTAL DESIGN

MATERIAL

The 80/100 penetration grade bitumen supplied by the Cenco Science, Malaysia was used as a control sample in this work. The physical properties of the base bitumen are presented in Table 1. The palm kernel oil-based monoester polyol (PKO-p) was produced from palm kernel oil (PKO-p) using the prepolymerization method and was added with 2,4-diphenylmethane diisocyanate (MDI) to produce polyurethane (PU). The PKO-p and MDI were supplied by the School of Chemical Sciences and Food Technology, UKM, Malaysia.

<table>
<thead>
<tr>
<th>Bitumen test</th>
<th>Standard Test</th>
<th>Bitumen Grade 80/100</th>
</tr>
</thead>
<tbody>
<tr>
<td>Penetration (0.1 mm) at 25 °C</td>
<td>ASTM D5</td>
<td>86</td>
</tr>
<tr>
<td>Softening point (°C)</td>
<td>ASTM D36</td>
<td>46</td>
</tr>
<tr>
<td>Ductility(cm) at 25 °C</td>
<td>ASTM D113</td>
<td>180</td>
</tr>
<tr>
<td>Viscosity (mPa.s) at 135 °C</td>
<td>ASTM D4402</td>
<td>332</td>
</tr>
<tr>
<td>Specific gravity (g/cm³) at 25 °C</td>
<td>ASTM D70</td>
<td>1.03</td>
</tr>
</tbody>
</table>

Table 1 Physical properties of base bitumen

SAMPLE PREPARATION

Samples were prepared by heating the bitumen at 145°C until it turns into liquid. At the same time, PKO-p and MDI were prepared at ratios of (1:0.6) and (1:0.8) by the weight of PU. The PKO-p was first blended with bitumen at 110°C at 2000 revolutions per minute for 15 minutes using a mechanical shear mixer. MDI was then added and blended using the same mixing parameter. A total of 9 samples were prepared and were coded as shown in Table 2. Consistency test, rotational viscosity test, and temperature susceptibility calculation were done on all samples to evaluate the effects of the PU with the different PKO-p and MDI ratios on virgin bitumen.

<table>
<thead>
<tr>
<th>Bitumen-PU (%)</th>
<th>Bitumen-PU (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Polyoil : MDI (1:0.6)</td>
<td>Polyoil : MDI (1:0.8)</td>
</tr>
<tr>
<td>100-0</td>
<td></td>
</tr>
<tr>
<td>95-5(a)</td>
<td>95-5(e)</td>
</tr>
<tr>
<td>90-10(b)</td>
<td>90-10(f)</td>
</tr>
<tr>
<td>85-15(c)</td>
<td>85-15(g)</td>
</tr>
<tr>
<td>80-20(d)</td>
<td>80-20(h)</td>
</tr>
</tbody>
</table>

Table 2 Sample proportion

3 LABORATORY TESTS

The effects of using different PKO-p, MDI ratios on the percentage of bitumen-PU were measured by conducting physical testing, namely penetration, softening point, ductility and storage stability tests, on each sample and the results were compared with conventional bitumen. Rotational viscosity (RV) test was conducted at different temperatures to determine the effects of bitumen-PU at elevated temperature; this analysis is usually used to estimate the mixing and compaction temperatures for a particular mixture design. This consistency tests is an important parameter in determining the suitable bitumen-PU ratio which have the same characteristics as the base bitumen. In addition, the temperature susceptibility of the modified bitumen was calculated in terms of PI by using the results obtained from the penetration and softening point tests. Another way to express the temperature susceptibility of bitumen is by measuring the PVN of the modified bitumen, where higher values of PI and PVN indicate lower susceptibility to temperature. The values for PI and PVN were calculated using Equations 1 and 2, respectively:

\[ PI = \frac{20(1 - 25A)}{1 + 50A} \]  

\[ A = \frac{\log penT1 - \log 800}{T1 - SP} \]
where \( \text{penT}_1 \) is the penetration value at 25°C, and \( \text{SP} \) is the ASTM softening point

\[
\text{PVN} = \frac{\log L - \log X}{\log L - \log M} \quad (1.5)
\]

where \( L = 4.25800 - 0.79670 \log P \), and \( M = 3.46289 - 0.61094 \log P \)

\( X \) is the viscosity in centistokes measured at 135°C, and \( P \) is the penetration value at 25°C

4 RESULTS AND DISCUSSIONS

PENETRATION TEST

The penetration test was done in accordance with the ASTM D5. The value of penetration depends on the hardness or softness of bitumen. This value is also used to indicate the temperature susceptibility parameter; i.e. PI and PVN. Figure 1 shows the results for the penetration test for PU blended with 80/100 penetration grade bitumen at ratios of (1:0.6) and (1:0.8) ratio. It shows that the ratio of (1:0.6) PKO-p to MDI has a higher penetration value than the (1:0.8) ratio, which indicates that higher MDI ratio produced harder bitumen. It was observed that for both ratios, the addition of 5% PU to the 80/100 penetration grade bitumen lowers the penetration value although the value is higher than the value for conventional bitumen. This shows that the addition of PU softens virgin bitumen by reducing its consistency. However, the trends for the penetration value for both ratios are the same and remain within the specification limit.

![Figure 1 Penetration value versus percentage of bitumen-PU (80/100)](image)

SOFTENING POINT TEST

The softening point test, which was conducted in accordance with the ASTM D36, measures the temperature at which bitumen begins to flow. Figure 2 shows the results for the softening point test for PU blended with 80/100 penetration grade bitumen at ratios of (1:0.6) and (1:0.8). The softening point values for both ratios of bitumen-PU are higher than that for the unmodified bitumen, except for the 5% PU. At this PU percentage, both ratios resulted in lower softening points of 0.3°C and 0.8°C, respectively compared to the unmodified bitumen. This could be due to the change in bitumen which became softer after modification and therefore lower the melting point. The general trend shows that the addition of PU increases the softening point of bitumen. The softening points for 10, 15 and 20% are within specification.
ROTATIONAL VISCOSITY TEST AT DIFFERENT TEMPERATURES

The viscosity of bitumen was measured at 20 rpm in accordance with the ASTM D4402 and the values were recorded at 110°C, 120°C, 135°C and 165°C, as shown in Figure 3. These viscosity values indicate the bitumen’s resistance to deformation of flow based on the internal friction of its molecules. At lower temperature, the viscosity of bitumen with the ratio of (1:0.8), which contains higher percentage of PU, is greater. The viscosity of the bitumen decreases as temperature increases. The increase in viscosity is a result of the hardening effect of the MDI. The dispersion of PU in base bitumen might have contributed to better bonding strength by resisting the flow of bitumen. The increase in viscosity results in greater stiffness at high pavement service temperatures, and this could also be beneficial in improving resistance towards rutting (Amirkhanian 2011 and Santaga 2012). These results show that, the unmodified bitumen has lower viscosity values than all bitumen-PU. This shows that increasing the percentage of PU has significantly reduced the penetration value whilst also increasing the softening point and viscosity of the bitumen.
TEMPERATURE SUSCEPTIBILITY

The PI and PVN were calculated to determine the effect of temperature susceptibility. Higher temperature susceptibility produces lower PI and PVN values. The PI and PVN values of the base bitumen and the PU-bitumen are presented in Table 3. Bitumen with a PI less than -2 is highly susceptible to temperature, and they usually exhibit brittleness at low temperatures, and are very prone to transverse cracking in cold climate (Read & Whiteoak, 2003). In this study, both PI and PVN show a general increasing trend with an increase with the percentage of PU, which resulted in reduced temperature susceptibility of the bitumen. Therefore, bitumen containing higher amount of PU has higher PI and PVN values, which could cause the asphalt mixtures to be more resistant to low temperature cracking and permanent deformation (rutting).

Table 3 PI and PVN of Unmodified bitumen and bitumen-PU

<table>
<thead>
<tr>
<th>Bitumen-PU (1:0.6)</th>
<th>PI</th>
<th>PVN</th>
<th>Bitumen-PU (1:0.8)</th>
<th>PI</th>
<th>PVN</th>
</tr>
</thead>
<tbody>
<tr>
<td>100-0</td>
<td>-0.9261</td>
<td>-0.67</td>
<td>100-0</td>
<td>-0.9261</td>
<td>-0.67</td>
</tr>
<tr>
<td>95.5(a)</td>
<td>-0.7306</td>
<td>-0.13</td>
<td>95.5(e)</td>
<td>-0.6325</td>
<td>0.14</td>
</tr>
<tr>
<td>90-10(b)</td>
<td>-0.5975</td>
<td>0.37</td>
<td>90-10(f)</td>
<td>-0.5500</td>
<td>0.88</td>
</tr>
<tr>
<td>85-15(c)</td>
<td>-0.4763</td>
<td>0.69</td>
<td>85-15(g)</td>
<td>-0.4038</td>
<td>0.98</td>
</tr>
<tr>
<td>80-20(d)</td>
<td>-0.0907</td>
<td>1.37</td>
<td>80-20(h)</td>
<td>-0.0759</td>
<td>1.51</td>
</tr>
</tbody>
</table>

DUCTILITY TEST

The ductility test was conducted at 25°C in accordance with the ASTM D113. Table 4 shows results of the ductility test for the unmodified bitumen and bitumen-PU. The ductility values for all bitumen-PU increase linearly with the addition of PU. This result is consistent with the results for the penetration test, where higher percentage of PU gradually increases the stiffness of the bitumen.

Table 4 Ductility of Unmodified bitumen and bitumen-PU

<table>
<thead>
<tr>
<th>Bitumen-PU (%) (1:0.6)</th>
<th>Ductility (cm)</th>
<th>Bitumen-PU (%) (1:0.8)</th>
<th>Ductility (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100-0</td>
<td>180</td>
<td>100-0</td>
<td>180</td>
</tr>
<tr>
<td>95-5(a)</td>
<td>90</td>
<td>95-5(e)</td>
<td>105</td>
</tr>
<tr>
<td>90-10(b)</td>
<td>112</td>
<td>90-10(f)</td>
<td>144</td>
</tr>
<tr>
<td>85-15(c)</td>
<td>135</td>
<td>85-15(g)</td>
<td>153</td>
</tr>
<tr>
<td>80-20(d)</td>
<td>141</td>
<td>80-20(h)</td>
<td>168</td>
</tr>
</tbody>
</table>

STORAGE STABILITY TEST

Storage stability test is a crucial test in determining the stability of a blend during transportation and application at plant. This test was conducted to ensure that the blending process is effective, and is done by comparing the softening point values of the top and bottom sections of the bitumen. Table 5 shows the comparison between top and bottom sections of unmodified bitumen and bitumen-PU. The results show that the difference between top and bottom sections of the aluminum tube for all samples are less than 2.2°C, hence the sample can be regarded as storage stable blend.

Table 5 Storage Stability of Unmodified bitumen and bitumen-PU

<table>
<thead>
<tr>
<th>Bitumen-PU (%) (1:0.6)</th>
<th>Temperature difference (°C) for top and bottom sections</th>
<th>Bitumen-PU (%) (1:0.8)</th>
<th>Temperature difference (°C) for top and bottom sections</th>
</tr>
</thead>
<tbody>
<tr>
<td>100-0</td>
<td>0.10</td>
<td>100-0</td>
<td>0.20</td>
</tr>
<tr>
<td>95-5(a)</td>
<td>0.70</td>
<td>95-5(e)</td>
<td>0.60</td>
</tr>
<tr>
<td>90-10(b)</td>
<td>1.10</td>
<td>90-10(f)</td>
<td>1.10</td>
</tr>
<tr>
<td>85-15(c)</td>
<td>1.40</td>
<td>85-15(g)</td>
<td>1.60</td>
</tr>
<tr>
<td>80-20(d)</td>
<td>1.80</td>
<td>80-20(h)</td>
<td>2.10</td>
</tr>
</tbody>
</table>
5 CONCLUSIONS AND RECOMMENDATION

The effects of different ratios of PKO-p and MDI with varying PU percentages were investigated in this study. As expected, when penetration value decreased, the values for softening point and viscosity increased; this indicates that the addition of PU enhanced the original properties of the bitumen. The RV value of bitumen-PU increases significantly with the addition of PU which indicates that the bitumen has high internal friction and resistance to flow. Based on the PI and PVN values, the bitumen-PU is considered to be within the classification of conventional paving bitumen. All PU-bitumen are storage stable blends as the difference between the top and the bottom sections did not exceed the permitted value. In conclusion, the ratios of (1:0.6) and (1:0.8) with 5 and 10% PU has good compatibility when compared with the base bitumen. For future research, it is suggested that smaller PU percentages between 2-10% to be selected and tested for aged conditions.

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REFERENCES

Federal Roads Network in the United Arab Emirates is acquiring importance and relevance along the country’s economy grows and develops. One key factor in the global economy is the transport section. The UAE Ministry of Infrastructure Development through the Roads Department is has as one of its main responsibilities to keep the current network within operational and safety conditions.

One of the main challenges this Department is affording, is to keep these operational conditions from a good design stage parallel to the extremely fast users’ growth (trucks specially). It seems that some of the factors determining durability have been overtaken by this fact and some of the typical procedures used previously needed to be changed or improved.

From the Roads Assets Management implemented in the Ministry durability conditions on the pavements can be determined and those segments out of the thresholds required from the Ministry can be detected. Some of those segments were improved to resist the tough conditions imposed by the heavy traffic by improving one component of the asphalt (the bitumen), by polymers.

PMB are widely used in the world, and in UAE we detected the need to include them as integral part of the execution of the works by defined stages but trying to accelerate its application.

This paper describes how the Roads Department included the PMB in some projects and what is the future vision of the Department regarding this important matter.
1. INTRODUCTION

The Roads Department in the Ministry of Infrastructure Development (before Ministry of Public Works) is responsible for the Maintenance and Conservation of the federal roads network. The main objective of the department is to keep a good level in the roads maintenance since safety and operational conditions of the network depend on them and those are considered the main indicators for the Department to satisfy the citizens demand (safety and comfort).

As part of the Federal policies to increase quality and safety UAE government the national agenda 2021 was launched. This agenda includes a set of national indicators in different sectors as education, healthcare, economy, police and security, housing, infrastructure and government services. All of them are long-term, measuring performance outcomes according to each one of the national priorities comparing UAE with global benchmarks. The performance of these KPI values will be established by periodically monitoring.

In order to improve those operational conditions, the maintenance performed by the Roads Department needs to have continuous evolution to need methods and procedures that make that maintenance more efficient in terms of life cycle (life span and cost).

One of the main issues to tackle are the side effects produced by the heavy trucks in the pavement. Those effects are mainly reflected in problems that affects safety for example rutting, raveling or bleeding.

A good knowledge of the source of these problems is a basic condition to know also what procedures need to be improved and how.

Below, an explanation of how Roads Department faced a continuous problem of severe rutting is explained. Description of the conditions and use of some known techniques helped to start a long path to a needed development in asphalt matters.

2. BACKGROUND. MINISTRY OF INFRASTRUCTURE DEVELOPMENT NETWORK.

Maintenance and operation of the federal roads is responsibility of the Roads Department. The network which Ministry of Infrastructure Development is in charge of, comprises 750Km of center line, 1,250 Km of carriageway and 3,500Km lane.

The roads included in that network are the federal ones, within the Emirates of Sharjah, Ajman, Umm Al Quaim, Ras Al Khaima and Fujairah.
Traffic conditions are constantly monitored by the MoID through a vehicle counting stations network (from these 25 stations, 20 of them are counting only and 5 are counting and weighting the vehicles).

3. FEDERAL ROADS OPERATIONAL CONDITIONS

Roads in UAE are subjected to a variety of different operational conditions which make the asphalt a complex work, trying to balance the most economical solutions along with the more durable techniques.

3.1 Traffic

The vehicle counting stations allocated in the roads network, from 2013 until now, have allowed to obtain some traffic information which is very valuable for monitoring some highly important traffic parameters.

One of these parameters is the growth of the vehicles within the network. In some places calculations indicate a high growth numbers, as an example were: 10% in road E-611 (Emirates road), or 8.5% in road E-88 (Sharjah – Dhaid – Masafi) from 2015 to 2016. Real examples of the traffic evolution in two of the main Federal Roads can be seen below showing per year the Average Daily Traffic (ADT) and the Heavy Weight Average Daily Traffic (HWADT).
Weigh in motion stations have measured axle weights of more than 20 tons in some cases, which will bring to the asphalt, transmission of high levels of horizontal stresses due to the turning or slowing down maneuvers from the vehicle itself.

Figure 5. Direct horizontal effects due to tangential acceleration in asphalt produced by heavy loads
3.2. Heat

This is one of the most important factors to take into account when designing asphaltic admixtures in the region. Heat affects the binder softening point, therefore the engineers in the MoID during the last years have performed a very good approach to the limits in bitumen content, arriving to maximum values of 3.8% contained in the asphalt hotmix. Temperatures of the asphalt surface during summer time have been measured, reaching sometimes values of more than 65 Celsius degrees.

Aging is a factor that will be strongly affected also due to the sun action.

3.3 Dust

This is not a factor itself affecting the rutting of the asphalt but has to do possibly with the effects of the sand in the aggregates polishing and adherence properties of the wheel with the asphalt. Values of Friction and Macrotecture are to be evaluated in the new stage for the Roads Assets Management contract, currently under development within the MoID.

4. SPECIFIC PROBLEMS

The above described factors created effects in some sections of the Federal Roads. These effects can be transformed into an operational problem which can create a safety issue.

Usually these effects appeared very soon after the execution of a new asphalt layers, specially in those roads carrying a huge amount of heavy vehicles. The increasing of the growing factor on heavy vehicles
During the last part of 2015, three sections within the Federal Network were detected having an accelerated of symptoms produced by the factors described above (specially deep rutting and bleeding). Those three sections showed these harmful effects with advanced condition, from 6 to 9 months after the execution of the new asphalt.

Those three sections and the description of the traffic crossing that cross section can be seen below:
These important issues affecting traffic condition and safety aspects demanded a quick and definitive action in order to increase and improve the life span of the asphalt layers.

5. ACTIONS FROM THE MOID

Once the needs were detected, some possibilities to fix these distresses were discussed in order to improve the material properties to resist all the acting effects (as mentioned above) using the shorter period of time (considering that few months back the same segment was closed different times due to the same problem).

The options discussed in the MoID to be applied as soon as possible were:

1) Polymer Modified Bitumen in the asphalt hotmix
2) Concrete pavement
3) Cement stabilized road base and asphalt

From all the options mentioned before it was decided to use the first one, since the other two seemed to required more execution time (preparation of the equipment, removal and replacement of road base or curing time).

At this stage to improve the timing approved by the Ministry of Interior for the traffic safety granted period, it was a must to perform the activities fast.

Preparation of a smaller milling depth and an lay of a new asphalt layer modified with bitumen seemed to be the best option, the most practical and the one providing larger expectations of the life span. The asphalt plant was ready to receive the modified bitumen and applied it on the mixture which was another factor to decide for this technique.

Below can be observed the characteristics of the admixture bitumen used:
### Grade: PG-76-22 - SBS Modified PMB

**Specifications**

AASHTO M320 / ASTM D6373 Table 1 Grade PG76-22

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Viscosity Pas at 135 C (AASHTO T316)</td>
<td>1.29</td>
</tr>
<tr>
<td>Dynamic shear, Original binder, G*/sin δ at 76 deg C (AASHTO T315)</td>
<td>1.75 Kpa</td>
</tr>
<tr>
<td>Dynamic shear, RTFOT Binder, G*/sin δ at 76 deg C (AASHTO T240+T315):</td>
<td>2.95 Kpa</td>
</tr>
<tr>
<td>Dynamic shear, RTFOT + PAV Binder, G*xsin δ at 31 deg C (AASHTO T240+R28+T315):</td>
<td>885 Kpa</td>
</tr>
<tr>
<td>Bending Beam rheometer (AASHTO T313)</td>
<td></td>
</tr>
<tr>
<td>Elastic recovery 84 % after RTFOT (ASTM D6084)</td>
<td></td>
</tr>
<tr>
<td>Ring and Ball softening Point (ASTM D36)</td>
<td></td>
</tr>
</tbody>
</table>

**Classified as: 76-22 - Continuous Performance Grade as: 79.5-25.8**

---

**Figure 11. Properties of the bitumen used in the rehabilitation of the asphalt layers**

**Table:**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Flash Point, AASHTO T 48, °C</strong></td>
<td>350</td>
</tr>
<tr>
<td>Rotational Viscosity, ASTM D 4402 @ 135°C</td>
<td>1.290</td>
</tr>
<tr>
<td><strong>Dynamic Shear Rheometer, AASHTO T 315 (Tmax = 83.2)</strong></td>
<td>1.00 kPa min</td>
</tr>
<tr>
<td>Temperature °C</td>
<td>G*/kPa</td>
</tr>
<tr>
<td>76</td>
<td>1.64</td>
</tr>
<tr>
<td>82</td>
<td>1.02</td>
</tr>
<tr>
<td><strong>RTFO (Rolling Thin Film Oven)</strong></td>
<td></td>
</tr>
<tr>
<td>% Mass Change, AASHTO T 240</td>
<td>0.033</td>
</tr>
<tr>
<td><strong>Dynamic Shear Rheometer, AASHTO T 315 (Tmax = 79.5)</strong></td>
<td>2.20 kPa min</td>
</tr>
<tr>
<td>Temperature °C</td>
<td>G*/kPa</td>
</tr>
<tr>
<td>76</td>
<td>2.65</td>
</tr>
<tr>
<td>82</td>
<td>1.61</td>
</tr>
<tr>
<td><strong>PAV (Pressure Aging Vessel), 100°C</strong></td>
<td></td>
</tr>
<tr>
<td>Dynamic Shear Rheometer, AASHTO T 315 (Tint = 20.7)</td>
<td>5000 kPa max</td>
</tr>
<tr>
<td>Temperature °C</td>
<td>G*/kPa</td>
</tr>
<tr>
<td>22</td>
<td>5890</td>
</tr>
<tr>
<td>19</td>
<td>8710</td>
</tr>
<tr>
<td><strong>Bending Beam Rheometer, AASHTO T 313 (Tm/ Stiffess: -17.3, MValue: -12.8)</strong></td>
<td></td>
</tr>
<tr>
<td>Temperature °C</td>
<td>Averages</td>
</tr>
<tr>
<td>-12</td>
<td>Stiffness, MPa</td>
</tr>
<tr>
<td></td>
<td>m-value</td>
</tr>
<tr>
<td>-18</td>
<td>Stiffness, MPa</td>
</tr>
<tr>
<td></td>
<td>m-value</td>
</tr>
</tbody>
</table>

**Figure 12. Properties of the bitumen used in the rehabilitation of the asphalt layers**
The characteristics of the asphalt hotmix used in the new asphalt layer are summarized below:

As an additional test, it was required from the PMB (Polymer Modified Bitumen), to provide results of test of a performance wheel test, of the previous mentioned hotmix admixture using the modified bitumen. To have an idea of the admixture performance for rutting.

### Bitumen

<table>
<thead>
<tr>
<th>Test Method</th>
<th>Test Results</th>
<th>Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>% of Voids at Reflow min 3%</td>
<td>1.85</td>
<td>3.44</td>
</tr>
<tr>
<td>% of Voids at Reflow max 3%</td>
<td>3.70</td>
<td>6.87</td>
</tr>
<tr>
<td>Rate of wheel tracking (μm/cycle)</td>
<td>0.031</td>
<td>0.057</td>
</tr>
<tr>
<td>Void content (%)</td>
<td>7.0</td>
<td>6.9</td>
</tr>
</tbody>
</table>

*60000 wheel passes or 30000 cycles

### PG 76-22

<table>
<thead>
<tr>
<th>Test Method</th>
<th>slab I</th>
<th>slab II</th>
<th>average</th>
</tr>
</thead>
<tbody>
<tr>
<td>absolute rut depth (mm)</td>
<td>1.76</td>
<td>3.17</td>
<td>2.47</td>
</tr>
<tr>
<td>proportional rut depth (%)</td>
<td>3.52</td>
<td>6.33</td>
<td>4.93</td>
</tr>
<tr>
<td>absolute rut depth (mm)</td>
<td>1.84</td>
<td>3.36</td>
<td>2.60</td>
</tr>
<tr>
<td>proportional rut depth (%)</td>
<td>3.68</td>
<td>6.71</td>
<td>5.19</td>
</tr>
<tr>
<td>absolute rut depth (mm)</td>
<td>1.85</td>
<td>3.44</td>
<td>2.65</td>
</tr>
<tr>
<td>proportional rut depth (%)</td>
<td>3.70</td>
<td>6.87</td>
<td>5.29</td>
</tr>
<tr>
<td>Void content (%)</td>
<td>7.0</td>
<td>6.9</td>
<td>7.0</td>
</tr>
</tbody>
</table>

* 20000 wheel passes or 10000 cycles
** 40000 wheel passes or 20000 cycles
*** 60000 wheel passes or 30000 cycles
The test was carried out in asphalt in two slabs of 5cm thickness. Both of them reached the target of maximum 4mm after 60,000 wheel passes at 60 Celsius degrees in water base.

6. EXECUTION AND RESULTS

The execution of the works was done during the last part of 2015 in the locations mentioned previously, and constant inspections have been performed since that time.

The milled layer from the existing asphalt had 6cm depth and the new wearing course will be 6cm using the PMB and hotmix asphalt described above.

The way it was possible to reach a bitumen rated PG76-22 was using polymers added to the bitumen. The option of adding polymers to modify the properties of the bitumen to reach the requested PG product was the most clear way to achieve the improvements required from the conventional bitumen to become admixture PG rated.

It was clear for us that the traffic was the most critical factor affecting the behavior of the asphalt, therefore, a correlation between the traffic and the corresponding PG should be founded and applied.
The sections and date in which the sections were executed are shown below:

![Figure 18. Location of the sections in which the solution was implemented](image)

Continuous inspection of the sections has been developed since that time. The solution has demonstrated to be highly effective, because despite any traffic conditions have been modified since then and considering that the weather, sand movement through the pavement and the materials are unchanged, then we can ensure now that the life span of the segments has been at least extended four times more than the usual life span for the same sections, before the application of the polymer.

7. CURRENT AND FUTURE EXPECTATIONS FROM THE MOID SIDE

As per the behavior obtained in the application of the PMB in the asphalt the MoID has found the need to request the use of this PMB at least in some sections of the future projects in new roads as well in roads maintenance.

The next steps will guide the MoID in the application, improvement and use as common practice of the PMB in hotmix admixtures for asphalt layers.

1) Organization of an explicative Seminar for the different parties intervening in the construction process as can be asphalt manufacturers, construction companies, consultants, laboratories and official stakeholders (the Seminar was entirely organized by the MoID under the title of Use of Polymer Modified Bitumen and Superpave Technology. The main aim of this Seminar was to inform the attendants about the PMB technology and PG rating (production and control), different experiences and case studies (presenting successes and failures and how were they corrected), steps towards a possible application of Superpave method and the general information of how the MoID will start to work in this line.

2) Announcement that the MoID will start asking the use of PG rated admixtures in asphalt layers in full projects or some sections of them, depending on the current demands of the design and use of the roads. The official first set of requirements are under study and will be incorporate to next projects’ TOR in 2018. The material characteristics defined until now are defined below:
3) In the short term, for the hotmix design at this stage it will be allow to design the admixture with the conventional Marshall method but applying the use of the PG in the binder part of the asphalt. The quality control will have two clear components, one for acceptance of the asphalt placed in the road (controlled mainly by the conventional parameters including the Marshall test), and the second one for the data collection (characterization of the material using the Superpave test).

<table>
<thead>
<tr>
<th>Item</th>
<th>Property</th>
<th>Test</th>
<th>PG76-22</th>
<th>PG76-16</th>
<th>PG70-22</th>
<th>PG67-22</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Saturated Fatigue Life (Ghorbani-Stoianovski Interactions)</td>
<td>ASTM D962</td>
<td>2.30</td>
<td>2.30</td>
<td>2.30</td>
<td>2.30</td>
</tr>
<tr>
<td>2</td>
<td>Workability, %T</td>
<td>ASTM D2402</td>
<td>3.00</td>
<td>3.00</td>
<td>3.00</td>
<td>3.00</td>
</tr>
<tr>
<td>3</td>
<td>Dynamic Shear, G'peak @ 70°C</td>
<td>ASTM D173</td>
<td>1.06</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>4</td>
<td>Separation Test, % Difference between G' @ 70°C and 10 cycles of Top and Bottom</td>
<td>ASTM D173</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>5</td>
<td>Time Stability, Average of 3 values measured in Separation test (item 6) divided by the initial G' value measured in item 5, range</td>
<td>0.8 - 1.2</td>
<td>0.8 - 1.2</td>
<td>0.8 - 1.2</td>
<td>0.8 - 1.2</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Permanent Retained on Screen Test, %</td>
<td>PWA 100</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>7</td>
<td>Stability, min, % by mass</td>
<td>ASTM D6960</td>
<td>60</td>
<td>60</td>
<td>60</td>
<td>60</td>
</tr>
<tr>
<td>8</td>
<td>Polymer Content, min, % by mass</td>
<td>ASTM D6960</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>9</td>
<td>Reclaimed Asphalt, min, %</td>
<td>ASTM D6960</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>10</td>
<td>Marshall Stability</td>
<td>ASTM D3672</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>11</td>
<td>Dynamic Shear, G'peak @ 70°C and 10 cycles, kPa</td>
<td>ASTM D173</td>
<td>2.20</td>
<td>2.20</td>
<td>2.20</td>
<td>2.20</td>
</tr>
<tr>
<td>12</td>
<td>Marshall Stability</td>
<td>ASTM D173</td>
<td>2.20</td>
<td>2.20</td>
<td>2.20</td>
<td>2.20</td>
</tr>
<tr>
<td>13</td>
<td>Superpave Recovery, R2 @ 70°C</td>
<td>ASTM D7045</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>14</td>
<td>Superpave Delay, R2 @ 70°C</td>
<td>ASTM D7045</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>15</td>
<td>AbsOLUTE Difference = Abs (∆T) [Top-Bottom] / ∆T</td>
<td>Time Stability = [Top-Bottom] / (2 from item 4)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 19. Required properties of the hotmix binder
4) Database evaluation (in the medium term), for all the data collected from the different contractors (as mentioned above). This database analysis will allow the MoID to know how the behavior of the asphalt is approaching to the hypothesis established by Superpave method.

5) If there is agreement about the acceptance on the application of Superpave as a possible technology in the roads, the MoID will give the guidelines to the contractors to design the admixture with Marshall conventional method or with Superpave technology. In any of both cases is expected that the MoID will request performance criteria for the asphalt behavior during the project’s life span.

The savings expected from the application of this stage are calculated, based in a minimum increase of 4 times the life span, considering the execution and the traffic diversion costs.

8. CONCLUSIONS

The roads contained in the Federal Network are exposed to several factors that accelerate the deterioration of the asphalts, mainly rutting and bleeding.

Some aspects will be difficult to control in the short term (for example the maximum axle weight or traffic volumes), therefore the only way to extend the life span of the pavements and increase the efficiency in maintenance is to improve the materials characteristics.

One successful way to achieve this objective is to strengthen the binder component of the asphaltic admixtures through the use of polymers. Allowing the admixture at the same time, to reach all the requirements to be catalogued as PG rated bitumen.

As per the internal experience of the MoID the use of PG rated admixtures, is one of the most efficient ways to make the asphalts to perform better and fulfill the needs of the national roads.

The MoID has a draft of how to proceed by steps in the implementation, study, analysis and implementation of better procedures and policies for the asphalt design as per the current needs demanded by the UAE Government to keep and improve the operational and safety conditions of the highways.
PAPER TITLE: An Optimization Model for Minimizing the Cost of Constructing Highway Vertical Alignments

TRACK: TS5.3

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KEYWORDS: Highway design; highway profile; highway vertical alignment; nonlinear optimization.

ABSTRACT:
Highway designers usually select the vertical alignment of a highway segment by creating a profile showing the actual ground surface and selecting initial and final grades to minimize the overall cut and fill quantities. Those grades are connected together with vertical parabolic curves. However, in many highway construction or rehabilitation projects, the cost of cut may be substantially different from that of fill. The parameters of the vertical curve include the initial grade, the final grade, the station and elevation of the point of vertical curvature (PVC), and the station and elevation of the point of vertical tangency (PVT). The values of those parameters are selected by a proposed non-linear optimization model to minimize the overall cost of cut and fill rather than to minimize the overall quantities of cut and fill. The proposed model is flexible to include any design constraints for particular design problems. Different application examples are provided using the Evolutionary Algorithm in Microsoft Excel’s Solver add-in. The application examples validated the model and demonstrated its advantage of minimizing the overall cost rather than minimizing the overall volume of cut and fill.
An Optimization Model for Minimizing the Cost of Constructing Highway Vertical Alignments

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1 INTRODUCTION

A highway vertical curve is typically a parabolic curve that is defined by the following parameters shown in Figure 1 (AASHTO 2011):

- The initial grade, \( g_1 \);
- The final grade, \( g_2 \);
- The location (station) of the point of vertical curvature (PVC), \( x_1 \);
- The location (station) of the point of vertical tangency (PVT), \( x_2 \);
- The elevation of the point of vertical curvature (PVC), \( y_1 \); and
- The elevation of the point of vertical tangency (PVT), \( y_2 \).

For designing a new highway, the designer needs to decide on the different values assigned to the six parameters shown above according to certain constraints and mathematical relationships that relate some of those parameters together. As for rehabilitation projects for existing highways, some of these parameters may have known values while other parameters may have unknown values due to soil consolidation and prior resurfacing and maintenance projects for the highway. Based on that, the elevation of the centreline of the proposed highway is usually measured, or computed, at different stations and the resulting profile usually has an irregular shape. The designer then would create the vertical alignment of the highway by visually fitting two straight lines and a parabolic curve to that existing land profile. This visual method is time consuming and would not minimize the cost of constructing this vertical alignment. To overcome the limitation of this visual method, an analytical method was developed (Easa et al. 1998) to accomplish the following two simultaneous tasks:

- To identify the start and end of linear (tangent) and curved segments automatically based on the trend of profile data; and
- To sequentially fit straight lines to the linear segments and clamped cubic spline functions to the curved segments

The limitation of the above method is that in lieu of the original parabolic curve, the method assumes that the profile follows a cubic-function spline curve, which has varying curvature rate and is more complex to analyse. Another method was developed to present a linear optimization model (Easa1999), which allowed the use of constraints for fitting straight lines and parabolic curves to highway profile data. In that case both the tangents and parabolic curves
were fitted simultaneously to the profile data, thus producing a better alignment than that of the spline curve. The only limitation in that method was that it considered locations (stations) of the start and end points of the parabolic curve \((x_1\text{ and } x_2)\) to be known. That assumption was essential for linear optimization model to limit decision variables to four variables \((g_1, g_2, y_1, \text{ and } y_2)\) subject to linear constraints. That model was later extended (Easa 2008) by utilizing a mathematical procedure involving three binary variables to model the discontinuities at the start and end of the vertical curve, which ultimately resulted in convergence to the guaranteed globally optimum solution. Another solution was introduced (Hu et al. 2004) to improve the above method by using least-square method instead of the linear programming optimization with assuming the start and end points of the parabolic curve \((x_1 \text{ and } x_2)\) to be decision variables; and therefore there would be six decision variables in total \((g_1, g_2, y_1, y_2, x_1, \text{ and } x_2)\).

In all the above methods, the objective function was to minimize the overall volume of cut and fill. However, in many situations, the unit cost of cut may be substantially different from that of fill. An example situation is where the soil is extremely hard so that blasting is needed to cut the soil. This paper proposes a new nonlinear optimization model that considers different unit costs for cut and fill. The objective function of the proposed model is to minimize the overall cost of cut and fill, which is calculated as the sum of the product of each volume (cut or fill) multiplied by the unit cost of that volume. Several application examples are also provided to illustrate how the model is used in different situations.

2 VERTICAL CURVE EQUATIONS

Given that \(g_1\) and \(g_2\) are the initial and final grades, respectively (positive for upgrade and negative for downgrade), and according to design guides (AASHTO 2011), the algebraic difference in grades, \(A\), for a vertical curve is given as:

\[
A = g_2 - g_1
\]

(1)

If the length of the vertical curve is \(L\), the rate of change of grade, \(r\), is calculated as:

\[
r = \frac{A}{L}
\]

(2)

The inverse of the \(r\) value, shown in Equation 2, is the \(K\) value, which is the length of the vertical curve needed to affect 1% change in the grade of the vertical curve. Since highway vertical curves are usually equal-tangent, the point of vertical curvature (PVC) and the point of vertical tangency (PVT) are usually located at equal distances from the point of vertical intersection (PVI). Assuming that the origin point is located at PVC, the offset, \(Y\), of a point at distance \(x\) is given by:

\[
Y = \frac{rx^2}{2}
\]

(3)

The corresponding elevation, \(y\), is given by:

\[
y = g_1x + \left(\frac{rx^2}{2}\right)
\]

(4)

By differentiating Equation (4) with respect to \(x\) and equating \(dy/dx\) to zero, the location of the highest (or lowest) point, \(x_{hl}\), can be shown to be equal to \((-g_1/r)\).

3 OPTIMIZATION MODEL

For a typical highway profile consisting of two grades connected by a vertical parabolic curve, the station and elevation of any point, \(i\), on the profile \((x_i \text{ and } y_i)\) can be computed from the coordinates of PVC and PVT that are \((x_1, y_1)\) and \((x_2, y_2)\), respectively. It is required to fit two tangents (grades) and a parabolic curve to the profile data. The range of the stations of the start and end of the parabolic curve \((x_j, x_2)\) can be specified based on the shape of the profile. The number of data points of the initial grade, the parabolic curve, and the final grade are denoted \(K, J, \text{ and } M\), respectively with total number \(N\) of data points. The exact locations of the start and end points, \((x_i, y_i)\) and \((x_2, y_2)\), and the initial and final grades, \(g_1\) and \(g_2\), are all decision variables that need to be computed. For any point, \(i\), the difference between the estimated and observed elevations can be given by:

\[
d_i = y_{ei} - y_{oi}
\]

(5)
In the above equation, $y_{ei}$ is the estimated elevation of point $i$ and $y_{oi}$ is the observed elevation of point $i$. For the initial grade, the estimated elevation, $y_{ei}$, of a point that has station $x_i$ is given by:

$$y_{ei} = y_1 - g_1(x_1 - x_i); \quad x_i \leq x_2$$  (6)

For the parabolic curve, the offset and the corresponding curve elevation of a point $(x_i, y_i)$, based on Equation (3) and Equation (4), are given by:

$$Y_i = r(x_i - x_1)^2 / 2$$  (7)

$$y_{ei} = y_1 + g_1(x_i - x_1) + (g_2 - g_1)(x_i - x_2)^2 / 2(x_2 - x_1); \quad x_i \leq x_i \leq x_2$$  (8)

For the final grade, the elevation, $y_{ei}$, of a point that has station $x_i$ is given by:

$$y_{ei} = y_2 + g_2(x_i - x_2); \quad x_i \geq x_1$$  (9)

If the elevations of the first and final points on the profile are both constraints, and to ensure linearity of the initial and final grades, the elevations of PVC and PVT may both be calculated as:

$$y_1 = y_{oi} + g_1(x_1)$$  (10)

$$y_2 = y_{on} + g_2(x_{on} - x_2)$$  (11)

In the above equation, the parameter $y_{oi}$ is the elevation of the first point on the profile, and the parameters $x_{on}$ and $y_{on}$ are the station and elevation of the last point on the profile. The positive values for the difference, $d_i$, indicate fill sections, and the negative values indicate cut sections. Let the variables $w$, $s_f$, and $s_c$ denote the road width, side slope at fill sections, and side slope at cut sections, respectively. Based on that, the total fill and cut volumes are calculated using the following equations (where $V_f$ is the total fill volume and $V_c$ is the total cut volume):

$$V_f = w \sum_{i=1}^n d_i + s_f \sum_{i=1}^n (d_i)^2 \quad [\text{where } d_i > 0]$$  (12)

$$V_c = w \sum_{i=1}^n d_i + s_c \sum_{i=1}^n (d_i)^2 \quad [\text{where } d_i < 0]$$  (13)

If the unit costs of fill and cut are $C_f$ and $C_c$, respectively, the total costs of fill and cut are calculated using the following equations (where $TC_f$ is the total cost of fill and $TC_c$ is the total cost of cut):

$$TC_f = V_f \times C_f$$  (14)

$$TC_c = V_c \times C_c$$  (15)

The objective function is to minimize the sum of the total cost of fill and total cost of cut:

$$\text{Minimize } [TC_f + TC_c]$$  (16)

To ensure an equal-tangent vertical curve, the following constraint must be added:

$$g_1(L/2) + g_2(L/2) = y_2 - y_1$$  (17)

Knowing that $L = x_2 - x_1$ and by re-arranging the equation, it can be reduced to:

$$2(y_2 - y_1) - (g_1 + g_2)(x_2 - x_1) = 0$$  (18)

The model is also subject to the following constraints:

$$x_1 > x_{oi}$$  (19)

$$x_2 < x_{on}$$  (20)

In the above equation, the parameter $x_{oi}$ is the station of the first point on the profile. To ensure adequate sight distance on the designed vertical curve, the following constraint must also be met:

$$|x_2 - x_1|/\left|g_2 - g_1\right| > K_{design}$$  (21)
In the above equation, the parameter $K_{\text{design}}$ is the minimum $K$ value required for the vertical curve to ensure adequate sight distance, which depends on the design speed, as given by geometric design guides (e.g. AASHTO 2011). To ensure that all decision variables have non-negative values, $g_1$ and $g_2$ may be replaced by two non-negative variables, such that:

\begin{align*}
  g_1 &= g_{11} - g_{12} \\
  g_2 &= g_{21} - g_{22}
\end{align*}

Equations (14 – 23) represent the nonlinear optimization model, which may be solved using any commercially available optimization software. More constraints may be added for particular vertical curve problems such as maintaining a certain height above an underpass or below an overpass or maintaining a certain elevation at a certain station for intersection with another roadway.

4 APPLICATION EXAMPLES

A hypothetical example is provided to validate the developed nonlinear optimization model. In the example, the given profile data typically represent that shown in Figure 2 without any error; and therefore the optimization value is expected to be zero. The elevations at different stations were calculated based on the parameters shown in Table 1. The example was solved using the Evolutionary Algorithm in Microsoft Excel’s Solver add-in. The decision variables calculated by the software were found to precisely match those shown in Table 1 with the optimization value found to equal zero, which validates the developed model. The example was solved again after the profile data have been deliberately altered to reflect a real-world situation where vertical curve elevations are changed as a result of soil consolidation and prior resurfacing projects. The example was once solved with assuming the unit costs of cut and fill to be $50/m^3$ and $30/m^3$, respectively. In that case, the feasible solution found by the optimization model calculated the quantities of cut and fill as 1.31 m$^3$ and 2.77 m$^3$, respectively. The total cost of cut and fill was $148.67. The example was solved again with assuming an extreme case of high unit cost of cut at $3000/m^3$ with keeping the unit cost of fill as $30/m^3$. In that case, the feasible solution was different and the quantities of cut and fill were 0.02 m$^3$ and 1.90 m$^3$, respectively, with total cost of cut and fill $1930.57. This substantial difference in the cut volume between the two examples is a result of the high unit cost of cut in the latter example, which demonstrates the advantage of using the developed model to minimize the overall cost rather than minimizing the overall cut and fill volumes. This objective is different from the objective of the previously developed models discussed in this paper that focused on minimizing the overall cut and fill volume. The profiles of the original data and the two examples are shown in Figure 3 for comparison purpose.
Table 1. Design parameters and decision variables for the validation example.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Design parameters:</strong></td>
<td></td>
</tr>
<tr>
<td>Width of roadbed (m)</td>
<td>14</td>
</tr>
<tr>
<td>Fill side slope (%)</td>
<td>3</td>
</tr>
<tr>
<td>Cut side slope (%)</td>
<td>1</td>
</tr>
<tr>
<td><strong>Decision variables:</strong></td>
<td></td>
</tr>
<tr>
<td>Initial grade ([g_1]) (%)</td>
<td>2.5</td>
</tr>
<tr>
<td>Final grade ([g_2]) (%)</td>
<td>-0.9</td>
</tr>
<tr>
<td>Station of PVC ([x_1]) (m)</td>
<td>140</td>
</tr>
<tr>
<td>Station of PVT ([x_2]) (m)</td>
<td>340</td>
</tr>
<tr>
<td>Elevation of PVC ([y_1]) (m)</td>
<td>4.947</td>
</tr>
<tr>
<td>Elevation of PVT ([y_2]) (m)</td>
<td>6.547</td>
</tr>
</tbody>
</table>

Figure 3. Profiles of the original data and the two application examples.

5 CONCLUSIONS

In this paper, a nonlinear optimization model was developed to select optimum vertical curve parameters for rehabilitation projects based on individual cut and fill cost items. The objective of the developed optimization model is to minimize the overall cut and fill costs rather than minimizing their overall quantities. The parameters selected by the optimization model include the initial grade, the final grade, the station and elevation of the point of vertical curvature (PVC), and the station and elevation of the point of vertical tangency (PVT). The model has the flexibility to include any constraints needed for particular design problems such as setting certain stations at certain elevations. The model also has the flexibility to include more specific cost itemization such as selecting different unit costs for different cut depths. Different application examples were provided using the Evolutionary Algorithm in Microsoft Excel’s Solver add-in, which validated the model and demonstrated its advantage of minimizing the overall cost rather than minimizing the overall cut and fill volumes. Although the model is designed for rehabilitation projects with a single parabolic curve, it can be extended to optimize more-complex profiles with multiple curves. The developed model can also be applied for designing new profiles by setting the appropriate design constraints.

ACKNOWLEDGEMENT
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REFERENCES

**PAPER TITLE**

ROAD ROUGHNESS & INVENTORY SURVEY WITH SMARTPHONES

**TRACK**

5.1 Preserving Road Assets

<table>
<thead>
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<th>ORGANIZATION</th>
<th>COUNTRY</th>
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<tr>
<td>Mikael KEDBACK</td>
<td>ASEAN Market Representative</td>
<td>Roadroid AB</td>
<td>Sweden</td>
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<tr>
<td>Lars FORSLÖF Basir HABIBI</td>
<td>Founder &amp; CEO Marketing Support</td>
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<td></td>
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</tr>
</tbody>
</table>

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**KEYWORDS:**

Road Inventory, road condition, asset management, smartphone, rural roads

**ABSTRACT:**

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, its fundamental to collect objective data about the road condition.

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.

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. Another smart sensor used in Roadroids solution for low volume roads is the Sensbit™ traffic sensor. This completely wireless sensor provides simple traffic counts and classifications even in remote areas. It runs on battery and measures AADT and classify vehicle types, speed etc.

Results shows that Roadroid has universal usage; but with a specific potential on Low-volume/Low Cost Roads (Rural Roads). The presentation will also show case studies, including data from an unpaved rural road in Sweden (LV734) where roughness surveys been made daily in combination of visual observations, temperature and smart sensor traffic counting.
ROAD ROUGHNESS & INVENTORY SURVEY WITH SMARTPHONES

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ABSTRACT

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, its fundamental to collect objective data about the road condition.

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1. ASSET MANAGEMENT & PRESERVATION

Building a road whether it is a 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.

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 rural and low volume roads. The system is easily accessible by easy to use technologies and devices and doesn’t require professional expertise to operate and carry-out a survey.

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 good 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 in a close to a real time road condition status platform where prompt information can be obtained any time.

![Figure 1. A dynamic road quality image of road network in Sweden](image)

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.
2. ROADROID MOBILE-BASED ROAD SURVEY SYSTEM – OVERVIEW

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). 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 on roads where heavy, complex and expensive equipment is impossible to use.

The Roadroid smart phone solution has two options for roughness data calculation:

(1) eIRI (estimated IRI)—based on a peak and RMS (root mean square) vibration analysis—which is correlated to laser measurements on paved roads. The setup is fixed but made for three types of cars and is thought to compensate for speed between 20~100 km/h. eIRI is the base for the RI (Roadroid index) classification of single points and stretches (road links) of the road;

(2) cIRI (calculated IRI)—based on the QCS (quarter-car simulation) [1] for sampling during a narrow speed range such as 60~80 km/h. When measuring cIRI, the sensitivity of the device can be calibrated by the operator to a known reference.

Research done by the University of Auckland in 20131 (Tarr, K. E. 2013) and the University of Pretoria in 20132 (Johnston, M. 201, Islam, T. 2013) showed that Roadroid system is consistent enough during IRI measurements with varying 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.

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. Over 100 parameters ranging from average road condition to detailed cracking inventories are available.

![FIGURE 3. Road inventory app being used to enter manual ocular inputs during a road inventory survey](image)

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.

![Roadroid Pro V2](image)

**FIGURE 4.** 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.

**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.

**Roadroid Traffic Count**

The traffic count app for android smartphones manually registers visual input of traffic flow. The app aims to replace pen and paper for temporary issues where automatic sensors can't be used. The app should be easier to use in the field, than pen and paper. The data presentation is done instantly with no interpretation of hand notes.

Typical user cases are those who need to verify and calibrate automatic traffic sensors, for temporary checks or for low volume roads.
Another smart sensor used in the Roadroid solution for LVRs is the Sensbit™ traffic sensor. This completely wireless sensor provides simple traffic counts and classifications, even in remote areas. It runs on battery and measures AADT and classify vehicle types, speed, etc.

**Wireless Traffic sensor + Roadroid**

- **Low installation cost!!!**
  - No electricity or data cables
- **Remote installation**
  - Install anywhere - needs only 3G.
- **Avoid damage/theft**
  - No visible road side installation.
- **Easy to use**
  - AADT in Roadroid - link to details.
3. CASE STUDIES

Switzerland - Periodic Monitoring of Road Roughness Condition

During 2016/2017, Roadroid has carried out a continuously daily survey of a low-volume gravel road in central Switzerland to monitor the variance of road condition during the whole year period and also to assess the seasonal effect on the road network. In this survey, the same road is surveyed everyday by a postman vehicle with Roadroid's application installed on an android smartphone mounted in the car. Within the survey, external factor i.e. temperature and traffic count is also taken into account. Moreover, a manual general assessment of the road i.e. road condition, potholes on the road, etc. is obtained by the surveyor too.

At the end, the results seem to be very logical and satisfactory. The output on the collected data by the Roadroid app well corresponds to the manual assessments and effects of external factors. This survey will help the road agencies to understand the seasonal and external factors effects on the road quality which will play a vital role while taking decisions on maintenance, sanctioning rules and regulations and safety measures.

Myanmar - Data collection on their national road network

Myanmar Ministry of Construction (MOC) used 5 Roadroid units to collect roughness data about their entire road network. The MOC roads vary from 2 lanes concrete roads, paved roads with different widths and own to flexible pavement DBST. There are also some gravel roads in the network. Except from roughness values, the spatial data collected can be used to build a road database.

For trimming and learning the adjustable constant for cIRI (calculated IRI), a test was made with 2 different cars and 4 units. Two units were mounted on left/right side of the cars to see how the cIRI constant changed.

Afghanistan - Data collection on regional gravel roads

In Afghanistan, Northern region, UNOPS runs a road project funded by SIDA. The project’s focus is gravel roads in the four provinces of Balkh, Sari Pul, Samangan and Jawzjan. Local partner is Public Works Department (PWD).

Roadroid was tested and evaluated during 2013 and then procured in late 2013. An onsite training held in June 2014 included a 3-day planning followed by a 6-day training for 15 road engineers.

A great challenge was the rough gravel roads, and the extreme type of damages. An extreme surface roughness is caused by the use of natural material, round stones mixed with sand. The sand gradually blows or wash away and the big round stones remain. • “Potholes” sometimes was bathtub size, needing creeping speed to pass. Its was not possible to classify that in terms of IRI.

4. 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).

Smartphone 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. 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 Roadroids 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, security threats in insecure areas and transportation costs where sometimes it really is a burden to transport survey vehicles and devices.

REFERENCES

CRACKING PERFORMANCE INVESTIGATION OF ASPHALT-RUBBER GAP-GRADED MIXTURES: EMPHASIS ON AGGREGATE GRADATION

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KEYWORDS:
Gap aggregate gradation, AR-Gap mix, cracking, fracture toughness, semi-circular bending test

ABSTRACT:
This study evaluated the effect of gap-graded aggregate gradations and binder content on the cracking resistance of asphalt-rubber gap-graded (AR-Gap) mixtures using static Semi-Circular Bending (SCB) test. Gap-graded aggregate gradations recommended by Arizona Department of Transportation or Arizona DOT (A), Texas DOT (B) and one newly proposed gap gradation (C) were considered to assess the effect of gradations. A total of 80 SCB specimens were utilized for experimental investigation encompassing nine AR-Gap mixtures. Fracture toughness ($K_{IC}$) of AR-Gap mixtures was determined at 0 and 25°C, and compared with a reference conventional dense-graded asphalt mix. At 95% confidence interval, the effects of gradation and binder content were found to be significant. Effect analyses revealed that higher fracture resistance can be obtained for gradation C at optimum binder content (OBC), whereas gradations A and B required higher binder content than OBC for higher fracture resistance at lower temperature. Further, gradation A showed higher fracture resistance at 25°C whereas gradations B and C produced higher $K_{IC}$ when higher binder content was used. In addition, fracture mechanism was investigated by graphical inspection of fundamental load-time relationship. Further, parametric indices provided insights into the relative occurrence of the time of crack initiation and post-crack initiation curves. Overall, it is envisioned that this study would help provide fundamental understanding in selecting the appropriate gap-graded aggregate gradation, and advance the state-of-the-art pertaining to fracture performance evaluation of asphalt mixtures.
Cracking Performance Investigation of Asphalt-Rubber Gap-Graded Mixtures: Emphasis on Aggregate Gradation

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1 INTRODUCTION

Cracking forms an elementary distress mechanism in low-temperature cracking and fatigue cracking of asphalt mixtures. Though the root causes of these two distresses utilize distinctly derived stress conditions, the continuum damage of both the processes can be well explained by the intrinsic fracture properties of the materials. Hence, a fundamental performance evaluation of asphalt mixture in respect of low-temperature and fatigue cracking must include a comprehensive understanding of fundamental cracking mechanism.

In the last few decades, extensive research (Krans et al. 1996, Mull et al. 2002, Wu et al 2005, Arahani and Ferdowsi, 2009, Huang et al 2009, Hassan and Khalid 2010, Biligiri et al. 2012, Wang et al. 2013, Im et al. 2014, Saha and Biligiri 2017) has been directed to evaluate the fracture properties of asphalt mixtures using fracture mechanics principles since it offers an in-depth knowledge of cracking mechanism over the conventional failure analysis. Majority of the research has quantified the cracking behavior of asphalt mixtures based on fracture parameters: fracture toughness ($K_{IC}$), fracture energy, and $J$-integral. Although the past literature explored and characterized the various factors that potentially contribute to the fracture resistance of asphalt mixtures, the mainstream literature of fracture in flexible pavements was mostly focused on the dense-graded (DG) asphalt mixtures.

In recent times, the application of asphalt-rubber gap-graded (AR-Gap) mixtures is well received and has become a popular technology across the world. Thus, an overall performance assessment of AR-Gap mixture necessitates a thorough understanding of its materials properties and their influences on cracking characteristics. Further, the practice of AR-Gap mixture was reported to be region-specific wherein each of the mix specification provided by highway agencies varies to comply with the expected cracking performance of a particular geographical scenario. In essence, a wide-range of mixture variables exists in the state-of-the-art pertaining to the design and analysis of AR-Gap mixture that chiefly includes, but not limited to: aggregate gradation and asphalt content. Therefore, there is a need to evaluate cracking performance of AR-Gap mixtures for two reasons: (a) quantify the effect of important mixture variables on fracture characteristics encompassing recommended specifications by different agencies, and (b) identify the cracking mechanism at varying mixture properties. With this background, this study investigated the cracking performance of AR-Gap mixtures in respect of aggregate gradation and asphalt content using static Semi-Circular Bending (SCB) test at different temperatures. The scope of the study included (Figure 1):

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure1.png}
\caption{Research framework}
\end{figure}
• Determination of fracture toughness for various AR-Gap mixtures using static SCB test
• Comparative assessment of fracture resistance with varying asphalt contents and aggregate gradations
• Statistical quantification and trend analyses of mix variables on fracture resistance at different temperatures
• Cracking mechanism investigation on the premise of load-time history during the specimen failure
• Analysis of behavioral response in the pre- and post-crack initiation regime using graphical inspection

2 MATERIALS AND EXPERIMENT PROGRAM

This study considered three types of AR-Gap mixtures prepared with commercially available crumb rubber modified binder, CRMB-60. Three gap-gradations included: Arizona gap-gradation recommended by Arizona Department of Transportation or Arizona DOT (Way et al. 2012) designated as gradation A, Texas gap-gradation recommended by Texas DOT (2004) designated as gradation B, and one newly proposed gradation designated as gradation C. Figure 2 presents the aggregate gradation curves. Superpave mix-design was employed to determine optimum binder content (OBC) for three aggregate gradations at $N_{des} = 125$ for traffic greater than 30 million standard axles (MSA). In this process, the OBC of gradations A, B, and C were 7.8, 5.8, and 5.4%, respectively. Next, Superpave gyratory specimens (height 170 mm and diameter 150 mm) were prepared at three binder contents: (OBC-1), OBC, and (OBC+1)%. Thus, a total of eighteen gyratory asphalt specimens were prepared for nine gap-graded asphalt mixtures at a rate of two specimens per mix type. Additionally, two DG asphalt specimens prepared at 4.5% binder content with viscosity graded VG-40 binder were also included for comparison purposes. Table 1 presents the summary of the mix designation used in this study.

Figure 2. Aggregate gradations

Table 1. Asphalt mixture designations

<table>
<thead>
<tr>
<th>Designation</th>
<th>Gradation</th>
<th>Binder</th>
<th>Binder content, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-1</td>
<td>A</td>
<td>CRMB60</td>
<td>OBC-1 = 6.8</td>
</tr>
<tr>
<td>A</td>
<td>A</td>
<td>CRMB60</td>
<td>OBC = 7.8</td>
</tr>
<tr>
<td>A+1</td>
<td>A</td>
<td>CRMB60</td>
<td>OBC+1 = 8.8</td>
</tr>
<tr>
<td>B-1</td>
<td>B</td>
<td>CRMB60</td>
<td>OBC-1 = 4.8</td>
</tr>
<tr>
<td>B</td>
<td>B</td>
<td>CRMB60</td>
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</tr>
<tr>
<td>B+1</td>
<td>B</td>
<td>CRMB60</td>
<td>OBC+1 = 6.8</td>
</tr>
<tr>
<td>C-1</td>
<td>C</td>
<td>CRMB60</td>
<td>OBC-1 = 4.4</td>
</tr>
<tr>
<td>C</td>
<td>C</td>
<td>CRMB60</td>
<td>OBC = 5.4</td>
</tr>
<tr>
<td>C+1</td>
<td>C</td>
<td>CRMB60</td>
<td>OBC+1 = 6.4</td>
</tr>
<tr>
<td>BC-VG40</td>
<td>BC-2</td>
<td>VG40</td>
<td>OBC = 4.5</td>
</tr>
</tbody>
</table>

All the Superpave gyratory specimens were sliced to obtain cylindrical specimens of 50 mm height and 150 mm diameter using a water-cooled diamond cutter. Then, the specimens were cut diametrically to produce semi-circular bending (SCB) specimens. In total, 72 SCB specimens were prepared for nine AR-Gap mixtures with eight replicates per mixture type. Additionally, eight SCB specimens of DG mix were also included in the experimental program. Air voids of all the specimens were checked prior to notching per AASHTO T166-10, and those specimens that did not fall in the target air void range ± 1% were excluded.

Total specimens = 72 SCB specimens (AR-Gap mixes) + 8 SCB specimens (DG mix) = 80 SCB specimens
3 EXPERIMENTAL INVESTIGATIONS AND RESULTS

The static SCB test was conducted on 80 specimens at 0 and 25 °C using a Universal Testing Machine (UTM) in accordance with AASHTO TP 105-13 standard. Fracture resistance of asphalt mixtures was measured using fracture toughness ($K_{IC}$) as the major assessor (Equation 1). SCB specimens and experimental setup are presented in Figure 3. The test specimens were conditioned for four hours prior to testing. Note that the test temperatures were selected with a view to evaluate cracking phenomena in respect of low-temperature cracking and fatigue cracking distresses. In the process of $K_{IC}$ estimation at 25 °C, the assumption of linear elastic fracture mechanics was considered to be valid as per the research conducted by Ozer et al (2016).

$$K_{IC} = \sigma_o \sqrt{\pi a} \times Y_1$$

Where:
- $K_{IC}$ = Fracture toughness, MPa. \(\text{vm}\)
- $\sigma_o$ = $P/(D\times L)$, MPa
- $P$ = Failure load, MN
- $D$ = Specimen diameter, mm
- $L$ = Specimen thickness, mm
- $a$ = Notch length, mm
- $Y_1$ = Dimensionless geometric factor = 4.782 + 1.219 ($\frac{a}{\delta}$) + 0.063 $e^{7.045(\frac{a}{\delta})}$

$K_{IC}$ of nine AR-Gap mixtures were determined at 0 and 25 °C and presented in Figure 4 and compared with the reference DG mix produced with VG40 binder. As expected, $K_{IC}$ reduced with increasing temperature. Interestingly, the variation in $K_{IC}$ of all the mixtures showed a similar trend at both temperatures indicating that the brittleness of the asphalt mixtures due to the change in temperature led to a linear change in the viscoelastic strength. With regard to the comparative variation amongst the AR-Gap mixtures, the mixtures prepared with gradation $C$: $C_{-1}$, $C$ and $C_{+1}$ resulted in the highest $K_{IC}$ than other mixtures. It revealed that gradation $C$ offered higher resistance against fracture crack propagation than gradations $A$ and $B$.

Further, the effect of the binder content was understood when AR-Gap mixtures were compared within the same gradation. In case of gradation $A$, $K_{IC}$ initially increased with an increase in the binder content and reached a maximum level at OBC, but $K_{IC}$ dropped with further increment in the asphalt content. Thus, it was inferred that maximum fracture resistance could be obtained at OBC for gradation $A$. On the other hand, gradation $B$ showed a little to no changes in $K_{IC}$ when binder content varied from (OBC-1) to OBC but a sudden increment was noticed at (OBC+1). Further, gradation $C$ showed a consistent increment of fracture resistance with increasing binder content, and $C_{+1}$ showed the highest $K_{IC}$ within the group. Thus, it is important to understand that although all the gradations followed gap-graded envelope, the effect of asphalt content was attributed to a distinctive fracture resistance of the mixture. This was because of the different forms of stone-to-stone contact that develop due to a change in the aggregate.
proportion in each gradation envelope. This observation set the platform to further investigate the effects of aggregate gradation and binder content on fracture properties.

![Fracture toughness of asphalt mixtures at: (a) 0 and (b) 25 °C](image)

Figure 4. Fracture toughness of asphalt mixtures at: (a) 0 and (b) 25 °C

### 4 RESULTS AND ANALYSES

#### 4.1 Analysis of Variance

Fracture resistance of nine asphalt mixtures was statistically analyzed to evaluate the effect of aggregate gradation and binder content at 95% confidence interval using MINITAB® statistical package. Statistical analysis was employed to examine the effect of two independent variables accounting for three levels of each one. As the first step of statistical investigation, two-tailed analysis of variance (ANOVA) was utilized and summarized in Table 2. ANOVA helped understand whether the mean of independent variables had significant influence of the dependent variable: $K_{IC}$. The hypotheses in this analysis were as follows:

- $H_0$: $\mu_{\text{Gradation A}} = \mu_{\text{Gradation B}} = \mu_{\text{Gradation C}}$ \& $\mu_{\text{OBC-1}} = \mu_{\text{OBC-1}} = \mu_{\text{OBC}}$
- $H_1$: At least one of the means is not equal

Note that the null hypothesis was rejected if the $p$-value of the factor showed a value less than 0.025 at 95% confidence interval. As observed in Table 2, both the factors estimated a $p$-value less than 0.025 indicating that the null hypothesis could be rejected. Hence, it was inferred that both aggregate gradation and binder content had significant effects on $K_{IC}$ on all the AR-Gap mixtures. Further, the interaction between aggregate gradation and binder content was also found to be significant at 25 °C indicating that the factors collectively produced an effect on the fracture resistance of asphalt mixtures.
Table 2. ANOVA summary

<table>
<thead>
<tr>
<th>Source</th>
<th>DF</th>
<th>SS_e</th>
<th>SS_adj</th>
<th>MS_e</th>
<th>MS_adj</th>
<th>F-value</th>
<th>P-value</th>
<th>Decision</th>
</tr>
</thead>
<tbody>
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<td>1.625</td>
<td>0.203</td>
<td>0.203</td>
<td>8.09</td>
<td>&lt; 0.025</td>
<td>-</td>
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</tr>
<tr>
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<td>1.167</td>
<td>0.583</td>
<td>0.583</td>
<td>23.23</td>
<td>&lt; 0.025</td>
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<tr>
<td>Binder Content</td>
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<td>0.221</td>
<td>0.11</td>
<td>0.11</td>
<td>4.4</td>
<td>&lt; 0.025</td>
<td>Significant</td>
<td></td>
</tr>
<tr>
<td>Interaction</td>
<td>4</td>
<td>0.237</td>
<td>0.059</td>
<td>0.059</td>
<td>2.36</td>
<td>&gt; 0.025</td>
<td>Insignificant</td>
<td></td>
</tr>
<tr>
<td>Total</td>
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<td>2.303</td>
<td>0</td>
<td>25</td>
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<td></td>
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</table>

<table>
<thead>
<tr>
<th>Source</th>
<th>DF</th>
<th>SS_e</th>
<th>SS_adj</th>
<th>MS_e</th>
<th>MS_adj</th>
<th>F-value</th>
<th>P-value</th>
<th>Decision</th>
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</thead>
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<td>0.104</td>
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<td>0.03</td>
<td>46.19</td>
<td>&lt; 0.025</td>
<td>-</td>
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<tr>
<td>Gradation</td>
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<td>0.097</td>
<td>0.05</td>
<td>0.05</td>
<td>86.76</td>
<td>&lt; 0.025</td>
<td>Significant</td>
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<tr>
<td>Binder Content</td>
<td>2</td>
<td>0.006</td>
<td>0.003</td>
<td>0.003</td>
<td>5.63</td>
<td>&lt; 0.025</td>
<td>Significant</td>
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<tr>
<td>Interaction</td>
<td>4</td>
<td>0.014</td>
<td>0.003</td>
<td>0.003</td>
<td>6.41</td>
<td>&lt; 0.025</td>
<td>Significant</td>
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<tr>
<td>Total</td>
<td>35</td>
<td>0.128</td>
<td>0</td>
<td>25</td>
<td></td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>

4.2 Effect Analyses

As the next step, main and interaction effects were analyzed to comprehensively understand the behavioral pattern of the response variables with respect to aggregate gradation and binder content. Figure 5 presents the main effect on $K_{IC}$ at $0^\circ$C and $25^\circ$C. As observed in Figure 5, $K_{IC}$ was found to be highest for gradation C, followed by gradations A and B at both temperatures. The relative increment in fracture resistance of gradation B was noticed to be higher at $0^\circ$C in comparison with $25^\circ$C. It explained that one could expect a relatively higher fracture resistance in fatigue performance for gradation B than that observed with regard to low-temperature cracking distress for the same gradation type. Further, the effect of binder content was found to be different at two temperatures. $K_{IC}$ increased with increasing binder content at $0^\circ$C whereas fracture resistance reduced when binder content increased from (OBC-1) to OBC, and then became insensitive with further increment in binder content at $25^\circ$C. It is important to note that the effect of binder content on all the gradations was statistically analyzed by averaging the influence of independent variables over all the gradations. Thus, it might not provide the accurate information that can be considered specific for each gradation type. In this context, interaction effect was better adjudged to explain the effect of binder content on individual gradation explicitly. Interaction analysis is provided next that details this system in respect of possible combinations of each independent variable.

Figure 6 presents the interaction effect of binder content and gradation on the response variable: $K_{IC}$ at $0^\circ$C and $25^\circ$C. As observed in Figure 6 (a), $K_{IC}$ increased for all the gradations when binder content was (OBC-1) and OBC at $0^\circ$C. Though $K_{IC}$ increased at (OBC+1), the increment in $K_{IC}$ for C at (OBC+1) was relatively smaller in comparison with gradation B. It indicated that higher fracture resistance can be obtained for gradation C at optimum level of binder content, whereas higher $K_{IC}$ can be expected when gradation B was blended with binder content higher than OBC. A slightly different observation was noticed for gradation A at both $0^\circ$C and $25^\circ$C where a reduction in $K_{IC}$ with increasing binder content was observed as shown in Figures 6 (a) and (b). This was possibly due to the fact that additional binder content did not help improve the adhesion between the aggregate and binder content, rather helped provide the lubrication along with the aggregate boundary. Further, gradation C exhibited the lowest $K_{IC}$ at optimum binder content, but it increased with further increment in the binder content. It indicated that an improvement in fracture resistance in the process of fatigue performance could be expected when binder content increased.

Figure 5: Main effect on fracture toughness at (a) $0^\circ$C and (b) $25^\circ$C
5. FAILURE BEHAVIOR INVESTIGATION

5.1 Load-Time Relationship

The next step towards understanding the fracture behavior of AR-Gap included the investigation of the fracture mechanism observed during the failure process. Though the loading rate of the experiment was kept constant for all the specimens, the resulting loading experienced by the specimens was found to be different. In this context, it is important to understand that the effective stress on the specimens that was measured by the load-cell did not reflect the total work-energy applied on the specimen. The energy loss during the development of fracture process zone (FPZ) at the initial stage of loading, and crack propagation at the later stage held the total energy balance in the system. Hence, the load-energy applied on the specimen. The energy loss during the development of fracture process zone (FPZ) at the initial stage of loading, and crack propagation at the later stage held the total energy balance in the system. Hence, the load-energy applied on the specimen. The energy loss during the development of fracture process zone (FPZ) at the initial stage of loading, and crack propagation at the later stage held the total energy balance in the system. Hence, the load-energy applied on the specimen. The energy loss during the development of fracture process zone (FPZ) at the initial stage of loading, and crack propagation at the later stage held the total energy balance in the system. Hence, the load-energy applied on the specimen. The energy loss during the development of fracture process zone (FPZ) at the initial stage of loading, and crack propagation at the later stage held the total energy balance in the system. Hence, the load-energy applied on the specimen. The energy loss during the development of fracture process zone (FPZ) at the initial stage of loading, and crack propagation at the later stage held the total energy balance in the system. Hence, the load-energy applied on the specimen.

Figure 6: Interaction effect on fracture toughness at (a) 0 °C and (b) 25 °C

Figure 7. Load-time relationship for three gradations at: (a) 0 and (b) 25 °C
The major differences in the fracture patterns at two temperatures were discerned with the insights of the loading history analysis. In case of 0 °C, all the three AR-Gap mixtures showed a rapid growth in load magnitude entailed by a sudden discontinuity at the propagation phase. It featured a brittle failure in which one can expect a sharp breakage/split of the specimens without any significant plastic strain. On the other hand, the specimens at 25 °C continued to consistently increase the loading magnitude until a peak was reached, and then gradually attenuated to a level of approximately 80% of the peak load. This failure mechanism demonstrated a close proximity to failure in which the viscous component of the mixture governed the failure process and arrested the crack before the complete breakage of the specimens. In this process, the specimens failed through slow but steady cracking process at the partial levels, and was identified by a load reduction experienced by the specimens.

With regard to the effect of aggregate gradations at a specified temperature, the load-time relationship of three mixtures prepared at OBC was analyzed with the help of graphical inspection. As observed in Figure 7 (a), gradation C showed a sharp increment with a higher peak load than other mixes. It indicated that although the gradation C offered a higher fracture resistance against failure, it reached the peak load faster than gradation B. Further, gradation B exhibited early occurrence of peak load than other two gradations indicating that one would expect early crack initiation for gradation B. Hence, it was inferred that other two gradations A and C might outperform gradation B in the field especially at low-temperature cracking.

Concurrently, the load-time relationships of all the gradations at 25 °C illustrated a similar trend with a subtle distinction from the observations made at 0 °C (Figure 7 (b)). In this case, the order of peak occurrence remained the same but the load-attenuation followed a completely different pattern. It was noteworthy that the targeted failure mechanism of asphalt mixtures at 25 °C employed a fatigue continuum where the viscoelastic damage continued to develop and eventually governed the gradual cracking through the thickness. As expected, the failure pattern of the specimens in the laboratory simulated crack initiation and gradual propagation phase with a clear demarcation made by the load-time relationships. Since both the phases are equally important for the fatigue evaluation, the performance assessment necessitated a comparative evaluation at both phases.

Though gradation A showed an early peak load, it retarded the crack propagation rate at a later stage. The other two gradations: B and C illustrated a different failure pattern in comparison with gradation A but similar to one-another without any significant difference. The difference in the OBC in conjunction with the effect of gradation attributed to the failure patterns of asphalt mixtures. Recall the OBC of three gradations: A, B, and C were 7.8, 5.8, and 5.4%, respectively. Higher binder content in the mixtures: A, A, and A imparted additional viscous effect that consequentially helped develop bigger FPZ and ensured a stable crack propagation. Though the binder content used in gradations B and C were different, similarities in fracture pattern can be noticed where both the mixtures showed early crack initiation and short crack propagation phase. It manifested the pronounced effect of aggregate gradation that nullified the impact of additional binder content in B, B, and B mixtures.

### 5.2 Graphical Analyses

Although the graphical inspection provided a good understanding pertaining to the failure pattern, an attempt was made to extract the parametrical measure of the load-time relationship that would help gain knowledge on the relative occurrence of crack initiation and post-crack initiation curves. Since the evaluation of fatigue process employed both the loading at pre- and post-peak period, the parametric measures were carried out for all the mixtures at 25 °C. Table 3 summarizes the graphical indices: time of peak occurrence ($T_p$), slope of the load-time curve ($m$), and intercept ($c_p$) for nine AR-Gap mixtures. Figure 8 schematically presents the significance of the parameters.

![Figure 8. Schematic of parametric indices](image-url)
Table 3. Parametric indices of load-time relationship at 25 °C

<table>
<thead>
<tr>
<th>Mixes</th>
<th>Tp (sec)</th>
<th>m</th>
<th>cp</th>
<th>R²</th>
</tr>
</thead>
<tbody>
<tr>
<td>A₁</td>
<td>46.48</td>
<td>-0.031</td>
<td>2.689</td>
<td>0.979</td>
</tr>
<tr>
<td>A</td>
<td>44.64</td>
<td>-0.025</td>
<td>2.014</td>
<td>0.982</td>
</tr>
<tr>
<td>A₂</td>
<td>43.52</td>
<td>-0.011</td>
<td>1.443</td>
<td>0.969</td>
</tr>
<tr>
<td>B₁</td>
<td>35.28</td>
<td>-0.064</td>
<td>3.319</td>
<td>0.960</td>
</tr>
<tr>
<td>B</td>
<td>28.80</td>
<td>-0.046</td>
<td>2.954</td>
<td>0.937</td>
</tr>
<tr>
<td>B₂</td>
<td>22.72</td>
<td>-0.038</td>
<td>2.297</td>
<td>0.941</td>
</tr>
<tr>
<td>C₁</td>
<td>36.72</td>
<td>-0.232</td>
<td>10.986</td>
<td>0.931</td>
</tr>
<tr>
<td>C</td>
<td>31.28</td>
<td>-0.875</td>
<td>5.426</td>
<td>0.952</td>
</tr>
<tr>
<td>C₂</td>
<td>27.72</td>
<td>-0.058</td>
<td>3.415</td>
<td>0.951</td>
</tr>
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</table>

As observed in Table 3, the mixtures prepared with gradation B exhibited lower $T_p$ as compared to other mixtures prepared with gradations A and C. Although the difference in $T_p$ among the mixture groups B and C were marginal, the combined effect of the gradation and binder content was identified by $m$ and $c_p$. As one can understand, higher $c_p$ indicates higher magnitude of peak-load whereas an increase in $m$ reveals a rapid fall in the load-time relationship. Thus, these three indices completely describe the characteristics of the load-time relationship. The asphalt mixtures prepared with gradation C showed the highest $m$ and $c_p$ exemplifying that the mixtures would experience sharp and rapid breakage once the crack initiation took place. Further, higher $c_p$ of the mixtures: C₂, C, and C₁ provided information about the peak load that was not quantified by $T_p$.

With regard to the effect of binder content on gradations, $T_p$ and $c_p$ increased with increasing binder content for all the gradations. It revealed that higher binder content in the asphalt mixtures prolonged the crack initiation and retarded the crack propagation rate at the post-crack initiation regime. Since the presence of additional binder content in the mix-matrix increased the potentiality of a higher degree of plastic straining around the crack tip, the mixtures plausibly developed bigger FPZ, which in turn reduced effective stress intensity. Further, the reduction in $T_p$ and $c_p$ was noticeably higher for the asphalt mixtures prepared with gradation C indicating that these mixes were more sensitive to change in the binder content. Thus, it can be concluded that the performance of these mixtures might be improved at the post-crack initiation phase when higher binder content was used.

6. CONCLUSIONS AND RECOMMENDATIONS

The objective of the study was to investigate the cracking performance of AR-Gap mixtures in respect of aggregate gradation, and asphalt content using Semi-Circular Bending (SCB) test. Three gap-graded aggregates were considered in this study and compared with a reference of one conventional DG mix. The significant contribution of this study included exploring insights of the fracture pattern of AR-Gap mixtures in respect of low-temperature and fatigue cracking using fracture mechanics principles. The major conclusions of the study are summarized as follows:

- **Fracture resistance determination**: $K_{IC}$ of nine AR-Gap mixtures were determined with four replicates at 0 and 25 °C using static SCB test. At 95% confidence interval, the effects of aggregate gradation and binder content were found to be significant. As expected, the fracture resistance of all the gradations decreased with increasing temperatures due to the reduction in the viscosity of the asphalt mixtures. Overall, the static SCB test was found to be promising in assessing the fracture resistance of AR-Gap mixtures in respect of low-temperature and fatigue cracking distress conditions.

- **Effect analyses**: revealed that higher fracture resistance could be obtained for gradation C at OBC, whereas gradations A and B required higher binder content than OBC to result in higher fracture resistance at low temperature. On the other hand, gradation A showed higher fracture resistance at 25 °C whereas gradations B and C produced higher $K_{IC}$ when higher binder content was used. Hence, gradation A was recommended to blend with lower than OBC for better fatigue performance whereas gradations B and C were expected to perform better in the presence of additional binder content.

- **Fracture mechanism investigation**: was carried out by graphical inspection of fundamental load-time relationship. All the AR-Gap mixtures at 0 °C showed a rapid growth in load magnitude that was entailed by a sudden discontinuity at the propagation phase. It was also understood that though gradation C offered the highest fracture resistance, these mixes reached the peak-load much faster than other two gradations. In contrast, all the gradations at 25 °C illustrated a load attenuation that featured a failure governed by plastic deformation.

- **Parametric indices**: provided insights to the relative occurrence of times of crack initiation and the post-crack initiation curves. Asphalt mixtures prepared with gradation C showed the highest $m$ and $c_p$ exemplifying that
these mixtures would experience sharp and rapid breakage once the crack initiation took place. $T_p$ and $c_p$ increased with increasing binder content for all the gradations. Hence, higher binder content in the asphalt mixtures prolonged the crack initiation and retarded the crack propagation rate in the post-crack initiation regime.

Even though this study investigated fracture resistance and failure mechanism of different gap-graded aggregate gradations, a similar methodology can also be employed to assess the effect of different asphalt binders and their impacts on these gradations. Future study is certainly needed in this direction to evaluate additional performance measures by using other mixture variables such as air voids and voids in the mineral aggregates. Albeit, it is envisioned that this study would help provide fundamental understanding in selecting the appropriate gap-graded aggregate gradation, and advance the state-of-the-art pertaining to fracture performance evaluation of asphalt mixtures.

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REFERENCES

ABSTRACT

- Title: Development and Deployment of Asset Management Systems applied to Ashghal (Public Works Authority of Qatar) Assets
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Development and Deployment of Asset Management Systems applied to Ashghal Assets

Knowing what you have, where you have it and what shape it is in are some of the fundamental requirements to effectively managing an established and quickly growing infrastructure.

In addition, Qatar’s 18,000 lane kilometers of assets is undergoing major upgrades and additions as part of its 2030 Vision. This paper will discuss the scope, strategies and methods currently in place and already deployed by Applus+ for Ashghal (Public Works Authority) with the aim of knowing when, where and what maintenance strategies should be undertaken to achieve the desired performance level within the given budget.

Several key projects already finished will be discussed. The projects relate to the following,

- Determining a comprehensive database of what exists. Development and assessment of high advanced asset inventory using lines, polilines and poligons through a 3D cloud point of the infrastructures.
- Determine the current condition by undertaking a comprehensive pavement condition survey that includes, surface condition (roughness, rutting, distress, friction) and structural assessment (FWD, GPR)
- 3D right-of-way video-log including 360° images.
- High accuracy mapping
- Deployment and installation of a web based asset viewer.
- Development and installation of a first class COTS PMS that provides different user levels for decision makers, engineering analysis, technical reporting, maintenance managers and more.

The paper will outline the various objectives, methodologies and tools that Applus+ have used to carry out the task at hand. Details of the latest 3D laser technology for distress analysis and asset inventory as well as the other technology that had been used will be discussed.
**INTRODUCTION**

In order to build, conserve and exploit a road network, it is required to know what you have and what condition it is in. This is accomplished by doing a complete asset inventory and pavement condition survey. A powerful asset and pavement management system to handle and analyze the huge quantities of data and ability to calculate the yearly budget needed to maintain the road network at the desirable level of quality and service is required.

In simple terms there are two categories of assets, pavement and non-pavement assets. Pavement assets typically include the pavement structure and bridges. Non-pavement assets encompass all other street furniture such as signs, hydrants, barriers and many more. The Doha Survey objective is to identify and collect information on all of Ashghal’s assets.

**ASSET INVENTORY: PAVEMENT**

Knowing the condition of the road is mandatory in order to determine proper maintenance actions and strategies. The current condition is determined by undertaking a comprehensive pavement condition survey using the latest high-tech devices for measuring surface condition (roughness, rutting, distress, friction) and a structural assessment (FWD, GPR).

![Surface Condition](image)

**Surface Condition**

Pavement roughness is measured using Greenwood’s LaserProf, a laser based profiler that can survey at traffic speeds up to 120 km/h. Profile is collected continuously with IRI (International Roughness Index) and PSI (Present Serviceability Index) provided for each 10 m interval.

Surface friction is measured continuously using the GripTester. A GN (Grip Number) is reported every 10 m. The GripTester can operate at speeds of 5-130 km/h, however, two different speeds are used for testing in Qatar, 50 km/h for the urban roads and 80 km/h for the high speed roads. A correlation equation is used to adjust the GN to an 80 km/h reference for the network.

Rutting is obtained from the RCMS (Road Crack Measurement System) instrumentation. Left and right ruts values as well as the average rut are reported on a 10 m interval. The rutting measurements are accurate to 1 mm.

![Surface Distress](image)

**Surface Distress**

Road surface distresses are determined using a Road Crack Measurement System (RCMS), a 3D laser profiling system from Pavemetrics. Data collection speeds up to 100 km/h are possible. The system will operate in daylight or darkness during the night time.
The RCMS captures 3D images of the road surface. JPEG images of range and intensity data provide 100% coverage of the road. Each image represents a 10 m x 4 m wide section of the road with 2000 profiles.

The intensity images are used to detect pavement markings and the ranges images are used to detect the surface distresses (and rutting).

An automated detection application is used to detected longitudinal, transverse and alligator cracks, rutting, raveling, micro-texture, potholes and ruts. A 100% manual review of the automated detection takes place to visually detect patching, bleeding, depressions and block cracking. 1 mm cracks can be detected with this system.

In addition to distress type, a severity level for each is determined based on various factors such as crack width, density and affected area.

Skid Resistance

Friction measurements are used to identify the skid resistance of the road surface – a measure of the ability of the road surface to facilitate safe braking distances. It can also help identify loss of micro-texture due to tire wear and destructive rubber removal methods such as hydro-blasting. The loss of macro-texture can also be evaluated and tracked.

The GripTester is the device of choice for this project. It is a continuous fixed-slip device system and operates within a speed range of 5 – 130 km/h. The most usual speed for highway testing is 50 km/h. In the State of Qatar, friction is measured at two test speeds: 80 km/h for high speed roads and 50 km/h for the lower speed roads. Testing at 50 km/h on the high speed roads is too dangerous and considered an unsafe practice. A correlation equation to provide a GN (Grip Number) to an 80 km/h reference speed is used. The correlation was obtained from empirical data collected at the start of the project.

FWD Data Analysis

In order to perform a proper analysis it is necessary to have a correct definition of the structure (materials, thicknesses of the different layers of the pavement), traffic information, pavement and air surface temperature etc. Dynatest’s ELMOD software is used to perform the analysis.

The back-calculation model considers the pavement as a multilayer system in which the material is elastic, linear and isotropic. In case of thin granular layers or similar modules, it is modeled in two layers together to ensure proper processing of the model. In the back-calculation we estimate a bowl of initial deflection from the input data for the different layers. This bowl is compared with the results from the measurements obtained in the trial. The ELMOD program calculates the values of the critical stress, strains and deflections for each layer and, knowing the fatigue laws, we can calculate the ESAL’s that produce the structural failure of the road. Comparing the ESAL’s with the traffic data of each road we will be able to know the remaining service life. In other words, we will now how many years will it take to the road to have a structural collapse in the case where we don’t do anything to maintain the road.

Structural Capacity

The Falling Weight Deflectometer (FWD) is used to access the road bearing capacity. The FWD is used to simulate the dynamic load of a truck. Each test consists of three test drops at different loads – 40 kN, 60 kN and 80 kN. The collection interval between drops varies by road classification. On primary and secondary roads measurements are taken at 100 m intervals in the
right-most (slow) lane. On selected local roads the interval is 500 m. The following parameters can be determined from the collected FWD data: dynamic E moduli, residual life, the critical layers and reinforcement needs. Because FWD is stop and go testing significant safety measures are taken when performing the tests. The use of TMAs (truck and trailer mounted attenuators), mobile variable message boards and safety cars are used to warn and protest the traveling public and the testing crews.

Ground Penetrating Radar (GPR)

The GPR system for the project uses two antennas allowing for the pavement and subgrade to be measured simultaneously. This multi-channel approach eliminates any necessary compromises of utilizing only one frequency, giving the highest possible resolution and information regarding each layer. The shielded antennas are mounted “in-line” on the MALÅ RoadCart and attached to the bumper of a vehicle via an ordinary tow hitch. The 1.6 GHz and 500 MHz ground-coupled antennas are positioned 1 cm and 4 cm above the pavement surface respectively. An encoder on the rear left wheel provides the triggering and distance measurements. Data can be collected at posted speed limits as high as 100 km/h with a data spacing interval of 20 cm.

The high frequency antenna allows us to estimate the layer thickness of the upper part of the road structure with 1 cm of resolution. The low frequency antenna allows us to know the thickness of the bottom part of the road structure. With the low frequency antenna we can obtain 3 cm of accuracy.

Layer thickness data from the GPR system is also used as input in the FWD data analysis. Additionally, because of the 2 m penetration of the radar signal there is potential to determine the possible presence underground utilities, however, this is more likely to happen when testing the 700 km of footpaths at slow speeds where the data interval is much less than 20 cm. GPR is also used to determine the presence of moisture and depth of the water table if it is within 2 m of the surface.

There is a significant challenge to obtaining the desired results. The ground structure composition provides an environment where the material is very conductive for the radar signal effectively absorbing it and minimizing any echo to be captured. We suspect that this is due to the high salt residue absorbed into the soil from the high water table. Special processing techniques are utilized to determine the sub-surface layer thickness.

ASSET INVENTORY: NON-PAVEMENT ASSETS

Data Collection

The first phase of completing the assets inventory is to run a survey vehicle that is able to capture the right-of-way image data. The image data consists of 360° video images and 3D LIDAR data.

The system used is a Trimble MX-3 with six cameras and 4 scanning lasers. Each camera has a 2 MP resolution (1600x1200). With the six cameras the system obtains the pictures used for the assets extraction. Pictures are synchronized with a high accuracy positioning system and the LIDAR point cloud. The positioning system is an Applanix POS LV 420. It is an inertial aided GPS georeferencing system. When the satellite signal is not available the equipment uses the inertial component to bridge the positioning gap. Even with one minute of satellite signal loss, for
example in urban canyon areas, the system provides an accuracy of better than 10 cm – X, Y. The system can operate at high speeds up to 100 km/h.

Asset Extraction
Using the geo-referenced images and point clouds it is possible to accurately locate (3D position) Ashghal’s assets to perform the comprehensive asset inventory. The inventory is more than just locating the asset; it includes recording specific attributes (as its shape through polilines and poligons drawings) of the asset and its condition. In most cases the condition will be determined from a more detailed manual inspection of asset. Where possible, from the image data, certain high level conditions will be found. The long term goal is collect all necessary information for the proper maintenance and operation of each asset.

Specific measurement tasks are quickly and economically solved with the help of the Trident and in house s/w. The Trident s/w performs automated detection and registration on some assets (signs, lane marking, pole detection, edge detection). Typical production rates are around 150 features per hour in when working in automated mode. Assets which are not possible to be collected automatically would be detected manually. It allows extraction the coordinates of the assets by de user. In manual mode, typical rates are around 30 features per hour.

Each asset is geo-reference with an accuracy of better than 0.3 m. Approximately 45 different asset types are being collected. Many asset types have attributes that define several specific assets. For example, the asset type STRUCTURES has an attribute that defines the asset as an underpass, overpass or tunnel. The asset is further defined by additional attributes that define the structures use as auto, rail or pedestrian. By setting up this type of relational structure it simplifies the extraction process and data organization, however, it masks the fact that over 100 different assets are being inventoried.

All data can be easily integrated into an ESRI shape file with comprehensive metadata for integration into GIS database.

Ashghal Assets
Basically, any asset owned, operated or maintained by Ashghal is included in the list of assets to be inventoried. Other assets are included in the survey because the information obtained is useful information for Ashghal with regard to pavement or information management. For example the location of bus stops and petrol stations is useful information with regard to understanding road conditions near these locations.

The amount of detail collected about each asset is determined by the need to know what information is needed to manage the asset in terms of maintaining its condition. This would include a list of parts and components contained within the asset in order to know what parts are needed if someone had to go out and repair it. The inventory data obtained from the Doha Survey is focused on the identification of the assets, the location and other attributes that can be obtained from the image data. The detailed component survey is carried out via manual hands-on inspections using portable collection tools.
DATA COLLECTION CHALLENGES

The environment of Qatar provides for some interesting challenges for the project.
- There are many significant road work projects in progress that make it a challenge to navigate the test equipment through the streets.
- The design and layout of the road network with regard to slip roads and service roads add significant driving overhead. It is not uncommon to have to drive 350+ km to obtain 20 km of test data.
- Without prior knowledge and information of the road network, planning the data collection routes for 10 pieces of test equipment is extremely difficult. Using Google maps to find roads and determine if they are roads to be inventoried is a challenge.
- Knowing what official permissions are required to collect is another challenge. You obtain what you think you need and wait to be stopped in order to find out what else is required.
- The extreme heat and testing conditions play havoc with the equipment. Data collection occurs in the evening in order to have the equipment operational, however, the LIDAR must work during the day because the system also included video cameras.
- New safety standards for work projects were being introduced in Qatar at the start of the project. These safety standards are a required step forward in the evolution of safe work programs, however, the equipment required (TMAs and VMS boards) is not readily available.

ASSET MANAGEMENT SYSTEM

Without the proper tools the data on its own is of little use. A comprehensive package to present, manage and analyse the data is needed. The basic premise of infrastructure asset management is to intervene at strategic points in an asset’s normal life cycle in order to extend the expected service life while maintain its performance. In order to ensure that premise, an Asset Management System allows Ashghal to know when, where and what maintenance strategies should be undertaken to achieve the desired performance level within the given budget. DTIMS is the package being used on this project.

In addition to data collected within this project additional data such as traffic counts, accidents and other information can be put into the system for use by the various modules of DTIMS. Most any information that the agency deems applicable to pavement and asset management can be added to the system as a source, it does not have to come from the survey data collected in this project.

DTIMS is a PMS (Pavement Management System) that includes asset management and map based data presentation capabilities. The features of the system are presented in the following sections.

Pavement Management

The mathematical models that define the evolution of parameters of pavements will be set by analyzing the information related to,
- Analysis of the PMS proposed by the World Bank (HDM-4).
- Experiences of the installation of road management expert systems.
- Other calculation models properly contrasted.

Evolution models predict the growth and decrease of structural and functional parameters of different network roads. Apart from considering the models that establish this evolution when maintenance work has not been carried out, they will also predict the evolution of these parameters when maintenance work has been executed.

One of the most important decisions to be made by the user of the pavement management system when posing a general problem is to define the desired condition level the
network should have. First it is necessary to specify if the whole network will maintain the same level or if it will be ranked by establishing different categories. The PMS enables the road network administration to establish the categories and name them. Once the Network has been divided, if applicable, the level of service for each category will have to be defined.

Whether the network is set to one category or multiple categories the first parameter to be set is the limit value to be maintained by this category. The value represents the desired trigger value for maintenance – a level at which maintenance activity is desirable.

The second limit value is the compulsory activity limit – the trigger value at which maintenance activity must take place. In other words, the road is not allowed to fall below this condition level.

The analysis of the information to determine the recommended maintenance activities includes all of the condition data, desired service and intervention levels, maintenance costs and strategies and the annual budget. The result of the analysis is a recommendation of the work to be performed within the budget and the cost for each work project. From the report it is possible to see the cost of each project, total cost, what projects are defined as recommended and what projects are defined as mandatory. The PMS will also show predicted future activities and associated costs based on historical and current condition data.

The PMS also provides the ability to perform a ‘what if’ analysis on the network or part thereof. The technical manager has the ability to manually select when work will happen, move projects in time, change the budget as well as add or delete projects as he sees fit. The end result would be a new analysis and report reflecting the changes made.

DTIMS provides a module that controls the daily work of the road maintenance team, trouble, revisions, working reports, pending and completed work, surveillance and weather trouble reports and reminders of periodic operations. The software allows for the schedule of work in the form of a list or calendar.

Cartography
The cartography is one of the main tools in DTIMS. It allows for cartographical surfing, using the typical tools of a GIS (Geographical Information System). In each of the modules of the management tool, users can make queries through the cartography interface.

Data Presentation
Any and all of the data is viewable in graphical or tabular form. ROW and pavement images are also available for viewing. Navigation through the system can be done through any of the views – the user can click on a particular location on a map or a point on a graph. The system will then navigate to that particular location for all data that is presented on the screen. Ease of use by different levels of users is paramount to a successful application. DTIMS satisfies that criteria.

Configuration and Setup
In order for the PMS to function certain data specific to Ashghal must be entered into the system. Ashghal’s maintenance strategies and associated decision criteria have to been defined within the PMS model. Standard costs for repair and maintenance activities are also required. This information is used to determine the specific maintenance strategy and its associated cost for any given section of road. Any decisions determined and put forth by DTIMS are only as good as the underlying data provided to it. Costing information should be as accurate as possible.

The large number of descriptive options within DTIMS enables users to view all the road data in many different formats. One of the best features of the system is the configuration of different levels of users. Users groups can be defined and specific report and data presentation formats can be preset to these groups. Restricting the capabilities of the specific group of users is desirable and achievable. In this manner you configure the system to allow viewers of the data to access and view only certain data elements. These groups could be divided into general users, safety engineers, maintenance engineers, administrators and
managements. It is clear that each of the defined groups would have different needs and access to different data and analysis functions.

As a web-based system tailoring the different reports and user groups is easily done. All the data is hosted by a local server and access maintained by the designated administrator. As a user logs in over the internet or intranet their profile is used to provide the proper functionality. In this manner even the casual user that may use the application infrequently does not have to re-learn or re-familiarize himself with it in order to effectively use it.

Configuration of the system and the custom reports for the various user groups is something that is done during system installation after consultation with Ashghal. This allows the system to present a list of available functions and reports to choose from instead of having to build it while online.

CONCLUSIONS

Throughout Applus works, Ashghal has a complete inventory of its road infrastructure and an objective assessment of its pavement. In addition to new construction and realignment projects underway Ashghal will have the required information to make informed decisions about the correct and economical maintenance strategies to maintain their roads.

As new and refurbished roads are completed it is vitally important to obtain the same information by performing a condition assessment as they come online. Without continuing the program on an annual basis it will be impossible to effectively manage the road network with any degree of objectivity and confidence. Decisions made are only as good as the information behind them.
A Proposed Traffic Safety Index for Macro-level Assessment of Local Rural Roads

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<th>PAPER TITLE</th>
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<td>Faculty of Engineering, Cairo University</td>
<td>Egypt</td>
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<td><a href="mailto:hassan.tahsin@cu.edu.eg">hassan.tahsin@cu.edu.eg</a></td>
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KEYWORDS:
Traffic safety, rural roads, safety index

ABSTRACT:
Little effort has been devoted to a systematic assessment of potential safety improvements on local rural roads such as those administered by most counties, some states, and certain federal agencies. Local road agencies need to have efficient investigation tools to identify safety concerns. The purpose of this study is to develop a Rural Road Safety Index (RRSI) to rank the road network according to the safety features and to identify the deficiencies in road sections. The proposed index will enable local road agencies need to identify, evaluate, and mitigate safety concerns. It will also help in solving road safety problems before they contribute to traffic accidents. The proposed index plays an important role for fund allocation for both state and local governments in order to gain the maximum benefit and return on investment. The paper presents a case study for a rural network in the county of Brookings, South Dakota. Results of the proposed safety index for a local rural network are presented and guidelines for enhancing safety on local rural roads are discussed.
A Proposed Traffic Safety Index for Macro-level Assessment of Local Rural Roads

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1 INTRODUCTION

According to the Traffic Safety Facts issued by NHTSA (1), of the 32,719 motor vehicle traffic fatalities in 2013, there were 17,696 (54%) that occurred in rural areas, 14,987 (46%) that occurred in urban areas, and 36 (<0.5%) that occurred in unknown areas. According to the 2013 American Community Survey from the U.S. Census Bureau, an estimated 19 percent of the U.S. population lived in rural areas. However, rural fatalities accounted for 54 percent of all traffic fatalities in 2013. Another study (2) found that the fatal crash incidence density was more than two times higher in rural than in urban areas. This was driven primarily by the injury fatality rate, which was almost three times higher in rural areas.

Reducing traffic crashes requires an understanding of causes and how the driver, the vehicle, and the roadway interact as a system. While road agencies have little control over drivers, vehicles, and weather, there are opportunities to make roads safer and more forgiving of driver errors. The greatest need of local governments is to improve the safety of their road network. The counties and townships have many miles of existing roadways that are in need of safety improvements with very limited funds. The use of reactive crash data to help identify the locations in need of safety improvements is marginal at best. This is primarily due to two factors. The first is that while crash rates are often highest for local roads on the basis of functional classification, the identification of potential spot improvement locations using this reactive tool is difficult. These crashes occur randomly and are generally not clustered as is typically the case with urban intersection data. The second factor is that many local crashes are often not reported in rural local areas (3). Given these facts, an analysis focusing on the safety issues of improvement is more appropriate. Another issue of importance in the low-volume rural road environment is that improving so many miles of roadway to current standards would be neither economical nor practical. For these rural local governments, a proactive program involving a functional classification of their rural roadway system is proposed.

Assessing the safety of a road requires knowledge of traffic (volume and type of vehicles) using the road and the speed they travel. Speed is an important consideration in roadway cross section width, horizontal and vertical curvature, driveway spacing, sight distance, and sign placement. Safe roads begin with designs appropriate for traffic, quality materials and construction, and good maintenance practices. To improve the safety of these roads, it is important to concentrate on areas where crashes have occurred or are likely to happen. These locations include curves, intersections, steep downgrades, and places where the road cross section changes, such as when a roadway narrows at a bridge.

The objective of this study is to develop a Rural Road Safety Index (RRSI), to rank the road network according to the safety features and identify the deficiencies in road sections. This RRSI will also help to solve road safety problems before they contribute to traffic accidents. The use of RRSI will produce a technique to support road safety reviews in order to quantify the safety gains that could be achieved by addressing the problems identified in the review process.

2 CHARACTERISTICS OF THE RURAL TRAFFIC CRASHES

Table 1 shows the distribution of South Dakota local rural roads crashes by collision classification for a 6 year period. About 44 percent of the crashes involve an animal struck by a vehicle, 25 percent rollover, 18 percent collision with fixed objects and 13 percent collision with other vehicles. There were no reported crashes involving a pedestrian on any of these local rural roads. Rollover crashes account for 42% of the deaths and injuries on local rural roads in South Dakota. About 30 percent of all crashes are collision with fixed objects, and 25 percent of the deaths and injuries because of collision with other vehicles.
### Table 1. South Dakota Local Rural Road Crashes

<table>
<thead>
<tr>
<th>First Harmful Event</th>
<th>% by all crashes</th>
<th>% by Injuries and Fatalities crashes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Motor Vehicle in Transport</td>
<td>13%</td>
<td>25%</td>
</tr>
<tr>
<td>Fixed object</td>
<td>18%</td>
<td>30%</td>
</tr>
<tr>
<td>An animal</td>
<td>44%</td>
<td>3%</td>
</tr>
<tr>
<td>Rollovers</td>
<td>25%</td>
<td>42%</td>
</tr>
</tbody>
</table>

Due to the random nature of most crashes on low volume local rural roads, and the fact that there are not high numbers or clusters of crashes, it is nearly impossible to predict crash locations with reliability. In order to make South Dakota rural local roads safer, the two leading types of fatal and injury crashes (run-off-the-road rollovers and collisions with fixed objects) need to be addressed through strategies to reduce roadway departure and to improve roadside safety.

Good surface condition, signing, pavement markings, delineation, and supereroded curves help drivers stay on the road. Traffic signs have standardized shapes and colors to help drivers recognize the message (4). Most of the single-vehicle, run-off-road crashes result from vehicles colliding with fixed objects (such as culvert ends, utility poles, trees, or unshielded bridge rail ends) or overturning while traversing slopes. A critical consideration is the clear zone—the area along the roadside that is free of fixed objects or dangerous slopes. The clear zone provides the driver an opportunity to recover and safely return to the roadway. The ground within the clear zone should be relatively flat and gently graded (not more than 4:1) (5). Rounded changes in slope will help a driver regain control of the vehicle and return to the roadway. Strategies for making roadsides safer include removing or relocating hazards to a place where they are less likely to be hit by vehicles leaving the roadway. Alternatively, reducing crash severity can be achieved by making roadside hardware crashworthy, making slopes traversable, and shielding fixed-object hazards with guardrail. For vehicles that do leave the roadway, the objective is to enable a driver to safely recover on the shoulder before encountering a fixed object or rolling over. Motorists do not purposely run off the road. The reasons for such events vary and include avoiding another vehicle, object, or animal in the travel lane; inattentive driving from inexperience, distraction, fatigue, alcohol, or drugs; the effects of weather on surface conditions; traveling too fast through a curve or down a grade. If a driver travels onto the roadside, the probability of a crash occurring can depend on roadside features such as the presence and location of fixed objects, shoulder edge drop-off, side slopes, ditches, and trees. A driver may be able to recover, or the severity of the crash can be reduced, if the roadside is fairly flat without objects. In addition, increased use of occupant restraints would provide a significant benefit in reducing the number and severity of injuries and deaths.

#### 3 FIELD VISITS AND REVIEWS

An important step in improving the safety performance of any road is a field review which focuses on roadway features contributing to or detracting from safety. Field studies were conducted on twenty-six local roads in Brookings County, South Dakota. All of the roads are low-volume rural arterials or collectors; access is permitted from residences and farms. All are unpaved roads; the right-of-way (ROW) is typically between 40-60 feet. Most of the sections have 30-55 mph speed limits. Average daily traffic volumes are between 50 and 400 vehicles per day. Three miles section of each road was driven; stops were made at each location where safety issues were identified or an accident had been reported. Numerous measurements of lane width, shoulder width, side-slope and cross-slope were taken in an effort to determine if any engineering treatments could be applicable. Table 2 shows the list of safety issues considered during the road safety field review visits. It is not intended to be all-inclusive, but can be used as a starting point. The purpose of a safety review is to flag features that need to be investigated further to determine what, if any, action should be taken. Judgment is needed in applying this list. There may be other factors identified during a review contributing to the safety of the road. The review list asks a series of questions to stimulate thinking about possible safety issues. It is formatted as a checklist with space for notes to be taken during a review in order to identify specific safety issues for possible further consideration.

The study team identified safety issues on most of the local roads where field reviews were conducted. The most common shortcomings are listed below:

- Due to the narrow roadway and curving alignment with numerous roadside obstructions or drop-offs, there are numerous locations where enhanced delineation could contribute to improving safety of the roads.
- There are several locations where the sight distance for entering traffic is restricted by trees, tree branches or bushes. Trimming or removing vegetation at these locations, especially where the entering angle of vehicles is not 90 degrees to through traffic would improve sight distance.
• There are locations where entering driveways have culvert ends with steep slopes creating an unsafe obstacle adjacent to the roadway. Flattening these slopes, in some cases by extending culverts, would improve safety for vehicles departing the roadway.

• There are several locations of mailboxes mounted on bases which are unsafe roadside obstacles. Consideration should be given to working with individual property owners to replace these installations with posts that will break away on impact.

• There were locations on reverse curves where Chevron signs could be added to emphasize and guide drivers through changing horizontal alignment. As a supplement to advance curve warning signs, Chevron Alignment signs in view of the motorist can effectively define the direction and sharpness of the curve.

• Vertical curves with narrow top and steep side slope make potentially high risk locations.

Table 2. Safety Questions List

<table>
<thead>
<tr>
<th>Item</th>
<th>Safety Questions</th>
</tr>
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<tbody>
<tr>
<td>Road Overview</td>
<td>Are there changes in land use and/or traffic or other environmental challenges such as terrain?</td>
</tr>
<tr>
<td>Crash History</td>
<td>Is there a history of crashes that points to specific problems areas?</td>
</tr>
<tr>
<td>Road Alignment &amp; Cross Section</td>
<td>How well does the roadway serve current and future traffic? Are lane width, shoulders, and sight distances adequate?</td>
</tr>
<tr>
<td>Roadside Features</td>
<td>Are there steep slopes, drainage features, narrow shoulders or clear zones, or fixed objects (narrow bridges, mailboxes, utility poles)? Are guardrails adequate and meet standards?</td>
</tr>
<tr>
<td>Gravel Road Surface Conditions</td>
<td>Are road surfaces well maintained (proper shape, smooth surfaces, loose gravel, or edge drop-offs)?</td>
</tr>
<tr>
<td>Paved Road Surface Conditions</td>
<td>Is surface smooth, adequate skid resistance, free of edge drop-offs?</td>
</tr>
<tr>
<td>Signing &amp; Pavement Marking</td>
<td>Are signs and pavement markings well maintained and meet the requirements of the MUTCD, including night time visibility?</td>
</tr>
<tr>
<td>Intersections &amp; Approaches</td>
<td>Are there sight restrictions (vegetation or other) that limit visibility? And is signing adequate?</td>
</tr>
<tr>
<td>Railroad Crossings</td>
<td>Are crossings properly signed and free of sight restrictions?</td>
</tr>
<tr>
<td>Pedestrians &amp; Bicycles</td>
<td>Are crossings clearly signed and marked, and are there areas of pedestrian activity (schools, playgrounds, parks) in need of special considerations?</td>
</tr>
<tr>
<td>Provisions for Heavy Vehicles</td>
<td>Are there operational issues due to the presence of heavy commercial or agricultural vehicles?</td>
</tr>
</tbody>
</table>

4 PROPOSED RURAL ROAD SAFETY INDEX (RRSI)

In order to place a meaningful index on how bad or good various sections of the local rural road network is, a value of some sort needs to be assigned. The safety effect is expressed by two indices, the estimated relative increase in accident injury, and the estimated relative increase in accident severity caused by various the safety issues. Each of the five major safety issues needs to be evaluated while indexing a road section. Each safety issue is graded on a full grade of four, where 4 being the best (no treatments are needed) and 1 is worst where some remedies are expected to be applied. Table 2 is used in determining the value of RRSI. In each section of the road, suggested to be 500ft, the review team scores the detailed safety issues according to the ranking system if the issue is present. Deduct points are added for each section and subtracted from 100 to determine the SSRI value for the section according to the following equation:

\[
RRSI_{\text{Section}} = 100 - \sum_{j=1}^{I} DP_j
\]

where \(DP_j\) is the deduct points for a safety issue \(J\). According to the site visits and investigation, the safety reviewers can calculate the RRSI for each road, create a safety ranking, and identify the safety priorities for the investigated network. RRSI is not the only method to estimate the safety of the local road; there are several other methods which can be coordinated along with this Index. With the help of RRSI, it will be easier for local governments, and other entities to prioritize the road safety and maintenance work.
Table 3. Traffic Safety Issue and Deduct Points

<table>
<thead>
<tr>
<th>Safety Issues</th>
<th>Deduct Points</th>
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<tr>
<td><strong>Roadside Obstacles</strong></td>
<td>30 Points</td>
</tr>
<tr>
<td>Rank 1 Rigid utility poles, obstacles, Inadequate bridge rails</td>
<td>≥ 5 times 30</td>
</tr>
<tr>
<td>Rank 2 Rigid utility poles, obstacles, Inadequate bridge rails</td>
<td>≥ 3 times 20</td>
</tr>
<tr>
<td>Rank 3 Rigid utility poles, obstacles, Inadequate bridge rails</td>
<td>≥ 1 time 10</td>
</tr>
<tr>
<td>Rank 4 No Roadside obstacles within the whole section.</td>
<td>0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>Signs and Delineation</strong></th>
<th>10 Points</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rank 1 Curve warning missing or ineffective on severe curve.</td>
<td>10</td>
</tr>
<tr>
<td>Rank 2 Guideposts or barrier reflectors damaged or missing.</td>
<td>7</td>
</tr>
<tr>
<td>Rank 3 Some Curve warning missing or inadequate.</td>
<td>5</td>
</tr>
<tr>
<td>Rank 4 No deficiencies on roadside signing.</td>
<td>0</td>
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<thead>
<tr>
<th><strong>Cross Section</strong></th>
<th>20 Points</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rank 1 LW &lt; 9 ft. / No shoulder.</td>
<td>20</td>
</tr>
<tr>
<td>Rank 2 9 ft. &lt; LW ≤ 10 ft /No shoulder.</td>
<td>15</td>
</tr>
<tr>
<td>Rank 3 9 ft. &lt; LW ≤ 10 ft. /Sufficient shoulder</td>
<td>10</td>
</tr>
<tr>
<td>Rank 4 LW &gt; 10 ft / Sufficient shoulder.</td>
<td>0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>Alignment and Accesses</strong></th>
<th>30 Points</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rank 1 Sight distance problems present</td>
<td>≥ 5 times 30</td>
</tr>
<tr>
<td>Rank 2 Sight distance problems present</td>
<td>≥ 3 times 20</td>
</tr>
<tr>
<td>Rank 3 Sight distance problems</td>
<td>≥ 1 time 10</td>
</tr>
<tr>
<td>Rank 4 No sight distance problems within the whole section.</td>
<td>0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>Road Surface and Maintenance</strong></th>
<th>10 Points</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rank 1 Very poor surface, corrugation and pot holes.</td>
<td>10</td>
</tr>
<tr>
<td>Rank 2 Presence of corrugation, pounding, holes.</td>
<td>7</td>
</tr>
<tr>
<td>Rank 3 Slightly deteriorated roads.</td>
<td>5</td>
</tr>
<tr>
<td>Rank 4 Good and very good surface</td>
<td>0</td>
</tr>
</tbody>
</table>

5 CASE STUDY OF A RURAL NETWORK

To apply the above concepts, a rural network in the county of Brookings, SD was selected. A total of 26 sites were selected. Some were at intersections while others were at midblock. After data collection, the values of the RRRI were calculated for each of the 26 sections and safety ranking was documented as shown in Table 4. Once the results are made available to decision makers, proper remedial measures could be applied. The road section with low score should be prioritizing for the safety concern. The main safety issues can be summarized as follows.

5.1 Roadside Obstacles

The main effect of roadside obstacle safety issues is not on accident probability but on accident severity. An unprotected culvert end next to the edge of the road presents a higher risk if located on the outside of a curve just over a hill than if located on a straight section of roadway in plain view of oncoming drivers. At several locations mailboxes are mounted on bases which are unsafe roadside obstacles. In South Dakota, total reported mailbox crashes have averaged 50 per year with an average of 10 per year reported on local rural roads. Consideration should be given to working with individual property owners to replace these installations with posts that will break away on impact. Roadside vegetation and limited lateral clearance to roadside objects presents a high potential for crashes, especially if close to an intersection or railroad crossing.
### Table 4. RRSI and Safety Prioritization Ranks

<table>
<thead>
<tr>
<th>Sec. No.</th>
<th>Segment</th>
<th>RRSI</th>
<th>Rank</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>214 Street West of I29</td>
<td>45</td>
<td>6</td>
</tr>
<tr>
<td>2</td>
<td>215 Street West of I29</td>
<td>53</td>
<td>9</td>
</tr>
<tr>
<td>3</td>
<td>203 Street West of I29</td>
<td>65</td>
<td>19</td>
</tr>
<tr>
<td>4</td>
<td>485 Avenue South</td>
<td>73</td>
<td>22</td>
</tr>
<tr>
<td>5</td>
<td>484 Avenue South</td>
<td>80</td>
<td>25</td>
</tr>
<tr>
<td>6</td>
<td>482 Avenue South</td>
<td>38</td>
<td>2</td>
</tr>
<tr>
<td>7</td>
<td>480 Avenue South</td>
<td>45</td>
<td>6</td>
</tr>
<tr>
<td>8</td>
<td>478 Avenue North</td>
<td>55</td>
<td>11</td>
</tr>
<tr>
<td>9</td>
<td>477 Avenue North</td>
<td>63</td>
<td>16</td>
</tr>
<tr>
<td>10</td>
<td>212 Street East of I29</td>
<td>35</td>
<td>1</td>
</tr>
<tr>
<td>11</td>
<td>209 Street East of I29</td>
<td>45</td>
<td>6</td>
</tr>
<tr>
<td>12</td>
<td>215 Street East of I29</td>
<td>55</td>
<td>11</td>
</tr>
<tr>
<td>13</td>
<td>203 Street East of I29</td>
<td>63</td>
<td>16</td>
</tr>
<tr>
<td>14</td>
<td>208 Street West of I29</td>
<td>85</td>
<td>26</td>
</tr>
<tr>
<td>15</td>
<td>206 Street West of I29</td>
<td>43</td>
<td>5</td>
</tr>
<tr>
<td>16</td>
<td>461 Avenue North</td>
<td>50</td>
<td>8</td>
</tr>
<tr>
<td>17</td>
<td>463 Avenue North</td>
<td>53</td>
<td>9</td>
</tr>
<tr>
<td>18</td>
<td>210 Avenue North</td>
<td>75</td>
<td>23</td>
</tr>
<tr>
<td>19</td>
<td>209 Avenue North</td>
<td>70</td>
<td>20</td>
</tr>
<tr>
<td>20</td>
<td>459 Avenue North</td>
<td>60</td>
<td>15</td>
</tr>
<tr>
<td>21</td>
<td>205 Street East of I29</td>
<td>55</td>
<td>11</td>
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<tr>
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<td>207 Street East of I29</td>
<td>40</td>
<td>3</td>
</tr>
<tr>
<td>23</td>
<td>480 Avenue South</td>
<td>40</td>
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<tr>
<td>24</td>
<td>477 Avenue South</td>
<td>55</td>
<td>11</td>
</tr>
<tr>
<td>25</td>
<td>475 Avenue South</td>
<td>70</td>
<td>20</td>
</tr>
<tr>
<td>26</td>
<td>476 Avenue South</td>
<td>63</td>
<td>16</td>
</tr>
</tbody>
</table>

### 5.2 Signs and Delineation

Traffic signs that have the greatest effect on traffic safety are warning signs. They call attention to unexpected conditions and situations that might not be readily apparent to road users. Faded and vandalized signs do not provide needed information to drivers, especially at night. Regulatory signs, such as speed limit and stop signs, can affect road safety by conveying essential information on safe behavior.

Delineation of the road is a critical safety issue especially at nighttime, or in snowy or rainy conditions. Supplementary delineation is an important safety factor in any condition; especially on horizontal curves and isolated curves with a short radius. A safety approach that provides long-range delineation of the roadway alignment is needed. The chevron alignment sign is an important traffic control device used to warn drivers of the severity of a curve by delineating the alignment of the road around that curve. Missing or ineffective chevrons and damaged or missing delineators or barrier reflectors can lead to an accident risk increase.

### 5.3 Cross Section

Roadway widths affect single vehicle, run-off-the-road and multiple vehicle, head-on, opposite-direction sideswipe and same-direction sideswipe accidents. Wider lanes and shoulder widths provide safer operation and fewer accidents. Inadequate sight distance on horizontal and vertical curves is a common accident contributory factor.

### 5.4 Alignment and Accesses

The geometry and location of direct accesses to roads can significantly increase accidents. The wrong location of access points (such as accesses on horizontal or vertical curves) can be very dangerous. Driveway location and density also have a dramatic effect of accesses on road safety.
5.5 Road Surface and Maintenance

Safe roads must provide uniformly sound surfaces and require good routine maintenance. Gravel roads need to be built with quality gravel, have the proper cross-section, and provide for adequate drainage. Consequently, study recommendations are intended to be as practical and cost-effective as possible while enhancing safety. For example, the traveled way at seven sites, where sharp crests exist, can be widened to the local roads standard with a motor grader by doing some aggressive shoulder work and should not require actual reconstruction with earthmoving equipment. Culverts under narrow roadways with steep side slopes create potential crash locations. Culverts should be extended to provide adequate clear zones and side slopes. Culvert ends that cannot be extended outside the clear zone should be marked with object markers.

6 CONCLUSIONS

The major issues that were identified by the study team were related to: intersections sight distance, angle of approach, signage, road alignment, vertical and horizontal curvature, culverts, table drains and location of signage, visibility and legibility of signs. If a non-frangible hazard on the roadside presents an unreasonable risk to road users, it should be considered for treatment. Acceptable treatments in descending order of preference include:

- Removal of the hazard to create a forgiving roadside environment.
- Relocation of the hazard.
- Replacement of the hazard with a more visible and frangible type.
- Shielding the hazard with safety barriers.
- Enhancing the visibility of the hazard.
- Warning road users of the hazard.

The study developed a Rural Road Safety Index (RRSI) in order to rank the road network according to the safety features and identify the deficiencies in road sections. Although RRSI is not the only method to estimate the safety of the local road; there are several other methods which can be coordinated along with this Index. With the help of RRSI, it will be easier for local governments, and other entities to prioritize the road safety and maintenance work.

REFERENCES

**PAPER TITLE**
Evaluation of Various Partial-Depth Repair Materials for Rigid Pavement

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**KEYWORDS:**
Traffic safety, rural roads

**ABSTRACT:**
One of the most disturbing and frustrating pavement distresses encountered by motorists is the potholes and spalls on concrete pavements. A priority for pavement engineers is to develop patching techniques, and to discover new quality materials to repair potholes and spalls on concrete pavements. The objective of this paper is to evaluate the performance of various types of patch materials for the repairs of potholes and spalls of concrete pavements. Laboratory compressive tests were first conducted on the composite specimens in order to determine the strength and bond of the patching material to concrete. Results were then used to determine which of the patch materials would be used in the performance testing. During testing, every distress is monitored closely by visual inspection for any signs of de-bonding near cracks, or a complete failure. The sum of load repetitions endured on the test track was used to equate the simulated life expectancy (SLE) of the materials. Recommendation and guidelines for the repairs of patching and spalls were established.
Evaluation of Various Partial-Depth Repair Materials for Rigid Pavement

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1 INTRODUCTION

There is a trend of growing traffic on interstate and urban highway systems today. The heavily traveled urban sections have begun to show effects of far exceeded traffic volumes, and environmental problems. Many Portland cement concrete (PCC) pavements are getting older and showing rapid deterioration. Cracks, spalls, and other distresses are occurring on most roadways. To maintain the serviceability of the highway systems, materials with advanced technology must be applied to repair these distresses.

Pothole and spall repair can be improved by using quality materials in conjunction with proper patching procedures. Traditional repair techniques of potholes and spalls required crews to square-cut the edges of the distress, which increase risk damaging of the original pavement in the vicinity of the repair. Extremely high intensity vibrations caused by the diamond blade saw may propagate cracks along the pavement. In addition, a 90-degree corner is very difficult for road crews, and over extending a saw cut on the corners occurs frequently. This leads to the initiation of cracking in the future. However, with new high strength patching materials the straight edge cutting of concrete may not be necessary. New concepts are needed to incorporate and apply recent advances in material technology to improve the performance and service life of highway pavements. Industry has produced many high-strength, fast-cure patching materials to enhance concrete pavement performance. Along with these materials, companies have been able to keep mixing and application procedures simplified. These fast set materials are able to minimize curing time allowing for less traffic congestion.

The objective of this research study was to evaluate the performance of patch materials for partial depth repairs of potholes and spalls from concrete pavements.

2 BACKGROUND

2.1 Partial-Depth Repairs

The American Concrete Pavement Association (ACPA) (1) reported on the guidelines for partial-depth repair. Partial-depth patches are placed to repair spalls either at pavement joints or at mid-slab locations. Spalls create a rough ride and can accelerate further deterioration. Spalling is a localized distress, and therefore, warrants a localized repair procedure for the pavement to be restored. Repair of this distress is needed to improve rideability, deter further deterioration, and provide proper edges so that the joints and cracks can be resealed effectively. It was learned that a good performance of partial-depth repairs can be obtained by:

- Limiting use of the technique to the top one-third of the slab and not extending repairs to a depth that allows the patching material to bear directly on dowel bars or reinforcing steel.
- Inserting a compressible material in all working joints and cracks or adjacent to the patch. The compressible material should extend 25.4 mm (1 in) below and 76.2 mm (3 in) laterally beyond patch boundaries.
- Using a bonding agent compatible with the selected patching material. Incompatibility will likely result in delaminating.
- Sealing the patch/slab perimeter interface using cement: water grout for cementitious patch materials to prevent moisture infiltration.
- Resealing the joint after repair to prevent water and incompressible from causing further damage.

Darter et al. (2) reported on the design guidelines of partial-depth patches. It was stated that the type of patch material to use for partial-depth patching depended on such factors as amount of time before opening to traffic, ambient...
temperature, cost, size, and depth of patches. The success of partial-depth patching depends on an adequate bond to existing concrete, therefore it is important that proper surface preparation to the concrete be done.

2.2 Performance of Patching Materials

Peshkin (3), as part of the Strategic Highway Research Program's (SHRP) initiative in highway operations area, studied the effectiveness, equipment, and procedures of partial-depth spall repair in Portland cement concrete (PCC) pavements. Test sites for the research were installed in highways across the United States and Canada. Installation and performance data was compiled and analyzed to provide preliminary indications about distress development and survival rates of various repairs. Under this project, 1,600 spalls were constructed with the cooperation of 15 different state DOT’s, 1 Canadian Province, and 1 city Department of Public Works. Private contractors installed two of the sites; while the sponsoring agency performed the rest of the work. Laboratory tests were performed on the repair materials and the data were used to identify correlation between laboratory test results and field performance. The research concluded that agencies might be able to save significant portions of their maintenance budgets and greatly increase the effectiveness of their repair activities by using higher quality materials.

Parker and Shoemaker (4) conducted studies on laboratory and field performance of three rapid-strength PCC pavement patch materials. The three selected materials were a rapid-setting PCC mixture, a rapid-setting fibrous PCC mixture, and Road Patch II, a proprietary material. The fibrous PCC contained discrete steel fibers. Laboratory mix designs revealed that PCC with and without steel fibers and the Road Patch II could produce an early compressive strength adequate for one-day patch construction. Laboratory tests showed that four-hour compressive strength tests of the PCC materials were higher than the proprietary material.

Ramey et. al. (5) reported on the strength and weathering characteristics of selected rapid-setting PCC pavement patching compounds. A laboratory testing program evaluated material and bonding properties, which are fundamentally related to the durability and performance of spall-type patches. Patching materials used for shallow-depth surface repairs of PCC pavements slabs and bridge decks were selected. Polymer concrete, Magnesium Phosphate Cement (MPC), Road Patch with steel fibers, and Epoxy/PCC were the four rapid setting materials chosen. Testing procedures included; compressive strength, tensile strength, direct shear, and impact tests. Three test series were employed to evaluate the effect of age and simulated weathering exposure.

3 LABORATORY TESTING PLAN

A total of sixteen composite specimens from the donated materials were cast in cylinders for compressive testing. The purpose of this test is to simulate conditions patching materials may exhibit during pavement rehabilitation. Various composite configurations are shown in Figure 1. Due to limited materials supplied by the manufacturers and the difficulty in casting the composite specimens, it was decided to only cast 1 or 2 cylinders per configuration. A simple technique for casting composite specimens may need to be studied in the future. The cylinders were standard 152.4 mm x 304.8 mm (6 in x 12 in) size. Type I PCC was used for the concrete part. The concrete cylinders were allowed to cure for 28 days in water baths before application of the patching materials. Thereafter, the five patching materials were prepared and arbitrarily placed in the ready-formed cylinder. The patching materials were alphabetically labeled on all composite materials. Materials D, E, and G were elastomeric concrete, while materials F and H were a cementitious mix. Although duplicate configurations were made, only one cylinder was cast with each composite material.

Results of the compressive strength tests are shown in Table 1. Good bonding and high compressive strength of the patching materials in composite specimens were the two criteria for selecting the materials to be tested for performance. Table 1 also shows the average compressive strength to be 48.5 MPa (7,034 psi) for the control sections. It is imperative to say that for the composite cylinders, the aspect ratio (LID) of the concrete part will not be equal to two as is for the ASTM standard cylinders. Therefore, the compressive strength for the composite specimens will be very difficult to predict. All patching configurations (PAC) numbers were taken from Figure 1. Materials D and G were both tested from PAC 1. The compressive strengths of material D and G only reached 7.32: MPa (1,061 psi) and 4.88: MPa (707 psi) respectively because of the early failure at the interface. However, composite cylinder G shows the cracks occurring at the interface and through the PCC under the compressive test. For this specimen, it was obvious that the patching material could sustain a higher load than the PCC but was still weak in bonding.
Figure 1. Composite Specimen Patching Configurations
### Table 1. Laboratory Compressive Test Summary Results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Config. No.</th>
<th>Load kN (lbs)</th>
<th>Strength Mpa (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control Specimen</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type I Cement; 28-day</td>
<td>----</td>
<td>874 (196,500)</td>
<td>47.9 (6,950)</td>
</tr>
<tr>
<td>cure</td>
<td></td>
<td>912 (205,000)</td>
<td>50.0 (7,250)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>867 (195,000)</td>
<td>47.6 (6,897)</td>
</tr>
<tr>
<td>Control Specimen</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type II Cement; 1-day</td>
<td>----</td>
<td>945 (212,000)</td>
<td>51.7 (7,498)</td>
</tr>
<tr>
<td>cure</td>
<td></td>
<td>923 (207,500)</td>
<td>50.6 (7,339)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>930 (209,000)</td>
<td>51.0 (7,397)</td>
</tr>
<tr>
<td>D</td>
<td>1</td>
<td>133 (30,000)</td>
<td>7.32 (1,061)</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>191 (43,000)</td>
<td>10.5 (1,521)</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>189 (42,500)</td>
<td>10.4 (1,503)</td>
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<tr>
<td>E</td>
<td>4</td>
<td>534 (120,000)</td>
<td>29.3 (4,244)</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>478 (107,500)</td>
<td>26.2 (3,802)</td>
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<tr>
<td></td>
<td>8</td>
<td>334 (75,000)</td>
<td>18.3 (2,653)</td>
</tr>
<tr>
<td>F</td>
<td>2</td>
<td>156 (35,000)</td>
<td>8.53 (1,238)</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>698 (157,000)</td>
<td>38.3 (5,553)</td>
</tr>
<tr>
<td>G</td>
<td>1</td>
<td>89 (20,000)</td>
<td>4.88 (707)</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>300 (67,500)</td>
<td>16.5 (2,387)</td>
</tr>
<tr>
<td>H</td>
<td>4</td>
<td>423 (95,000)</td>
<td>23.2 (3,360)</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>778 (175,000)</td>
<td>42.7 (6,189)</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>756 (170,000)</td>
<td>41.5 (6,013)</td>
</tr>
</tbody>
</table>

Material D also tested on PAC 5 and 8 shows an interesting fracture of the concrete, but the concrete portion surrounding the patching material remained in its cylinder shape. The compressive strength of both specimens only reached approximately 103 MPa (1,500 psi). This may be due to the higher aspect ratio of PCC while the patching material was confined by the concrete. According to Mohr’s failure theory, the material can sustain a higher load with application of a confining pressure. This may be the reason why the concrete failed before the patching materials. Three other composite samples were tested with material E from PAC’s 4, 6, and 8. The compressive strength of composite cylinders was 29.3 MPa (4,244 psi), 26.2 (3,802 psi), and 18.3 MPa (2,653 psi) respectively. The fractures took place along the composite specimen. Since PAC 4, 6, and 8 were partially filled with patching material E, no de-bonding was permitted. A confining pressure contributed by the surrounding concrete may have increased the failure strength of the patching materials. Therefore the fracture would take place in the PCC and not in the patching material. PAC 4, 6, and 8 demonstrated this actually happening. The test further hinted that the high strength patching material could simply fill a typical shaped pothole without worrying about de-bonding.

PAC 2 and 6 cylinders were cast with material F. The compressive strength was 8.53:MPa (1,238 psi) and 38.3 :MPa (5,553 psi) respectively. PAC 2 clearly failed in bonding, which resulted in a low compressive strength. The entire specimen fractured through patching material F and the PCC. This occurrence is evident because of the higher compressive strength of 38.3 MPa (5,553 psi). This material certainly would not be chosen for the accelerated performance test due to its poor bonding characteristics although it had a relatively high compressive strength. Material G was tested with PAC 1 and 3 cylinders. PAC 1 failed in bonding quickly at a very low compressive strength. For PAC 3, the aspect ratio was approximately 1 for the PCC. However, the compressive strength only reached 16.5 MPa (2,387 psi) when the concrete cracked. Due to this result, it is recommended that more samples be cast and tested in the future. Three composite specimens were cast with material H, in PAC’s 4, 6, and 7 cylinders for compressive tests. The compressive strength reached approximately 41.4MPa (6,000 psi) for PAC’s 6 and 7, closely achieving the compressive strength of the control sections. However, PAC 4 only strength may be attributed to the conical shaped composite specimen, which allowed the higher strength patching material due to the confining pressure to wedge into the lower strength PCC causing failure in the PCC. Photographs 15 and 16 show the interesting fracture patterns of the laboratory samples. The final compressive tests were conducted on the fast-set Type II cement mix. As previously mentioned the Type II cement mix was used for the control section during the accelerated performance testing the two cylindrical specimens were allowed to cure for only 24-hours. Yet, the average compressive strength for the Type II cement mix was 51.2 MPa (7,419 psi)

### 4 TEST FACILITIES

The facility (7) comprises a test track 15.2m (50ft) in diameter, a variable weight-loading apparatus, and a power source. The current track surface is a 1.8m (6ft) width concrete pavement supported on an earth embankment. The loading system consists of three supports of 7.6m (25ft) long W36x150 steel beams spaced from a pivot at 120-degree intervals. Each support beam is attached to a hydraulically driven dual-wheel truck-axle assembly. A water tank 3.7m (12ft) in diameter by 2.4m (8ft) height is centrally mounted on top of the support beams and is used to create
additional weight to the loading system. The total weight of the loading apparatus and water can vary between 134kN (30,000lbs) and 356kN (80,000lbs) and is evenly distributed to the three dual-wheel assemblies. The entire loading machine is powered by a 220hp diesel engine with a hydraulic transmission and is capable of driving up to 48km/hr (30mph) speed in either clockwise or counter-clockwise direction. A center support assembly, used to hold the entire system in place, is designed to restrain the testing machine from horizontal movement while allowing free rotation, vertical movement, and a small amount of tilt. The facility used to simulate actual traffic loads applied to all tests, was a dual wheel loading of 44.5 kN (10,000 lbs). During testing, every distress is monitored closely by visual inspection to detect if any signs of de-bonding, wear, cracks, or complete failure have occurred. The sum of load repetitions endured on the concrete pavement will then be used to equate the simulated life expectancy (SLE) of the materials.

4.1 Pothole Construction on the Test Track
Figure 2 illustrates the test track with pothole layouts. Half of the track was employed to test the performance of patching materials for pothole distresses. It was proposed to have fourteen such potholes to be used in the accelerated performance test. Three different pothole sizes were created: seven 304.8 cm x 30.48 cm x 12.7 cm (1 ft x 1 ft x 5 in) (Detail 1), seven 60.96 cm x 30.48 cm x 12.7 cm (2 ft x 1 ft x 5 in) (Detail 2), and two feathered edged 304.8 cm x 30.48 cm x 127 cm (1 ft x 1 ft x 5 in) (Detail 3). As stated earlier, the other half of the test track was still being tested from a previous project. The concrete pavement was constructed to a 25.4 cm (10 in) in thickness for artificial pothole creation.

4.2 Application of Patch Materials
The placement of the patch materials onto the pre-formed potholes was not an easy task for this project. The manufacturer's instructions for placing the patching material were read carefully, noting all necessary materials and equipment to be used. The material had a flush finish with the vertical edge of pavement at the joint. The patching material was placed immediately after mixing and it was placed at once instead of in stages. The patch was finished from the center of the patch out to the edges so to maximize bonding capacity between the patch and the existing pavement, especially near the top of the patch.

Levelling the materials with the track surface became somewhat difficult with the cementitious materials and the control section. The addition of 6.8 kg (15 lb) to 11.3 kg (25 lb) of aggregate to the cementitious materials made for a fairly dry mix. The mix design for the fast-setting Type II cement required a 67 -stone aggregate, thus the control section with became particularly difficult to place and finish properly. The volume of the aggregate also became a hindrance when levelling the patching material to the surface of the test track. Characteristics of the patching materials are summarized in Table 2.

### Table 2. Characteristics of the Patching Materials

<table>
<thead>
<tr>
<th>Material</th>
<th>Property</th>
<th>Mixing procedure</th>
<th>Workability</th>
<th>Curing time (hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Polymer</td>
<td>Moderate</td>
<td>Moderate</td>
<td>2-4</td>
</tr>
<tr>
<td>B</td>
<td>Cementitious</td>
<td>Easy</td>
<td>Moderate</td>
<td>2</td>
</tr>
<tr>
<td>C</td>
<td>Cementitious</td>
<td>Easy</td>
<td>Moderate</td>
<td>4</td>
</tr>
<tr>
<td>D</td>
<td>Elastomeric</td>
<td>Easy</td>
<td>Easy</td>
<td>4</td>
</tr>
<tr>
<td>E</td>
<td>Elastomeric</td>
<td>Moderate</td>
<td>Moderate</td>
<td>4</td>
</tr>
<tr>
<td>H</td>
<td>Cementitious</td>
<td>Easy</td>
<td>Moderate</td>
<td>2</td>
</tr>
<tr>
<td>Control Section</td>
<td>Type II (PCC)</td>
<td>hard</td>
<td>Difficult</td>
<td>24</td>
</tr>
</tbody>
</table>

5 LIFE EXPECTANCY AND TEST OBSERVATIONS
A total of 500,000 load repetitions was applied. Although this quantity of repetitions is considered low, noticeable fatigue to some materials is present. Some stresses were observed in two materials. The 500,000 repetition represented a simulated life expectancy of 17.7 years, assuming an ADT of 10,000 with an average percent of heavy trucks of 6%.

Material D, and E, which are Elastomeric Concrete, was failing due to de-bonding. It was noticed that after placing material D, shrinkage quickly occurred in the morning showing cracks along the interface of the material and concrete slab. However, the separations due to de-bonding were not seen during the warmer times of the day. In case of Material E, de-bonding failure has occurred due to the fatigue testing. Shrinkage/expansion cracks along Material E’s surface were present. This was due to excessive addition of water while mixing the cementitious material. This factor was evident, because the other pothole with Material E present had no cracks or other failures associated with bad mixing procedures.
6 CONCLUSIONS

The results of the compressive strength tests indicated the configurations used to cast the composite specimens might be reliable in choosing the most dependable materials. All of the samples tested in the laboratory showed unique fracture planes and bonding characteristics with the concrete. Failures occurred between 7.32 MPa (1,061 psi) and 42.7 MPa (6,189 psi) depending upon the material, the patching configuration and the aspect ratio (L/D). The results of the fracture patterns and compressive strength data have been evaluated and conceptually analyzed. Many types of quality materials exist in today's market, and to select the most appropriate specifications and guidelines must be made for all manufacturers' to follow. The UCF-CATT facility was used to simulate actual traffic loads applied to all tests, was a dual wheel loading of 44.5 kN (10,000 lbs). During testing, every distress is monitored closely by visual inspection to detect if any signs of de-bonding, wear, cracks, or complete failure have occurred. The sum of load repetitions endured on the concrete pavement will then be used to equate the simulated life expectancy (SLE) of the materials. After a total of 500,000 load repetitions of 44.5 kN (10,000 lbs) wheel load applied at this test facility, some signs of patching
distress have been found. The patches were observed and inspected daily, thus knowing the time any damage had
incurred. At the end of the testing schedule, the 500,000 repetition represented a simulated life expectancy of 17.7
years, assuming an ADT of 10,000 with an average percent of heavy trucks of 6%. Some de-bonding failures have
occurred. These failures occurred because of shrinkage due to weather changes, and de-lamination of the patching
material from the concrete interface. Elastomeric and polymer materials particularly had shrinkage problems. Cracking
also occurred in a cementitious mix due to excessive water in the mix design. However, failure from severe wearing,
cracking, and spalling has not been observed of any of the patch materials. The two feather-edged potholes, which are
simulating realistic pothole conditions on the highway, have performed well to this point. This has proven to be
valuable, because if crews do not have to square-cut partial-depth patches, placing the material directly into the distress
can save time and money. However, in determining the pavement life extension, both the life of the individual repair
and the life of the pavement as a whole shall be accounted for. The life-cycle cost analysis will be based upon
application of the patching material on an actual state highway requiring maintenance, or one with available
maintenance data. The costs of each material will be determined and evaluated with its service life. This evaluation will
then be compared to the present materials used in the rehabilitation of concrete pavements. The life-cycle cost analysis
for each method can then be studied to determine the best alternative for partial-depth patching. This investigation is an
important step in convincing engineers and agencies that there are many reliable materials for the use in today’s overly
traveled interstate and highway systems.

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of Central Florida.
Division.
Concrete Pavement Technology Center, Iowa State University, Institute for Transportation,
### PAPER TITLE
Performance behavior of bitumen modified by new generation of activated rubber powder

### KEYWORDS:
Activated rubber powder; dynamic viscosity; storage stability; complex shear modulus; irreversible shear compliance; activating catalyst; PPA

### ABSTRACT:
This paper presents results of more than 5 year innovation development which is related to improving modification of bitumen by crumb rubber. The traditional crumb rubber coming from old tires is further pulverized and activated using high-speed disintegration process during which fractal rubber particles of size < 0.8 mm are gained and used as a modifier together with suitable additives in bitumen. A performance improved and storage stable product is finally generated. Such modified bitumen in several alternatives was tested using traditional empirical tests as well as performance-based (functional) characteristics focusing mainly on the deformation behavior. Technically most efficient rubber modified binders where then used in some asphalt mixtures to verify their added value to the performance of the asphalt. Selected results are presented and discussed in the paper focusing on bituminous binders containing 10-15 % of the activated rubber powder.
Performance behavior of bitumen modified by new generation of activated rubber powder

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1 INTRODUCTION

From the literature (Marcant & Martin 2009) the yearly waste production of old tires worldwide reaches more than 120 million tires of different types and composition. This represents more than 7,500 kilotonnes of used rubber most of it coming from the EU countries and North America. More waste tires are expected to be produced in the coming years considering the fast economic growth taking place in many Asian countries. Regulations with respect to disposition of waste tires vary from country to country. Since several years back, it is forbidden to dispose waste tires in landfill in most developed regions. However, in some parts of the United States, slashed tires can be land filled, though and other solutions are preferred. The European waste management strategy recommended its recycling and reuse. Asphalt pavements and bitumen modifications are for several decades seen as a potential field for crumb rubber utilization.

Crumb rubber can be utilized in the form of dry or wet process. In the dry process, the crumb rubber is added directly to the asphalt mixture as a modifier and substitute to part of the finer aggregates. In the wet process, the crumb rubber is blended wet with the bitumen and subsequently modifies the bitumen properties. The scope of work of this paper refers to the wet process. One of the crucial issues related to this type of bitumen modification is the homogeneity of the final crumb rubber modified binder (CRMB). It is possible to produce CRMB directly in an asphalt mixing plant (continuous blend) and there are various solutions of bitumen blenders which are applied for such production method. The question always prevails if this is the most suitable solution and if homogeneity is secured as well as sufficient flexibility. If CRMB is produced industrially in a refinery or in a bitumen manufacturing plant (terminal blend), quality control should be better and the properties of the final product can be assured. In case of EU market each producer is responsible also for all necessary steps related to European REACH directive (registration of chemical compounds used on the market). The target then is to get a binder which can be transported for longer distances and ideally can be stored in a mixing plant for a few days. A CRMB that is homogeneous yet stable when stored over extended period of time is required. This is not usually easy to attain due to the presence of very strong sulphur bonds in the rubber. To improve the storage stability, several approaches based on polyphosphoric acid (PPA), macrocyclic polymers and others, have been tried. Usually the additive itself is not the solution and the combination of CR-bitumen-additive works only for limited rubber content.

With the above premises, this paper describes the use of special type of disintegration technique for producing pulverized rubber with particle size < 0.8 mm. This paper also analyses selected types of catalysts and additives which might help to produce a storage stable product. The process of high speed grinding (disintegration) follows the procedures described e.g. in (Valentin et al. 2013).

2 STUDY OBJECTIVES

This study aims to assess the characteristics of CRMB binders the preparation of which involved mechanically activated, fine-ground (pulverized) rubber with maximum particle size of 0.8 mm, as well as a new type of benzothiazol-based activating catalyst. In parallel some designs contained instead of this catalyst poly-phosphoric acid. The objective was to prepare such CRMB binder composition that would, besides improved functional characteristics, achieve a storage-stable (i.e. homogeneous) modified bituminous binder. Both components were applied to standard distilled straight-run bitumen. In two cases, modified binders were tested where the binder had been mixed with the rubber already in the course of the grinding process as such
An option of CRMB binder with poly-phosphoric acid was designed for the sake of comparison as well.

3 SCOPE OF EXPERIMENTAL ASSESSMENT

The experimental CRMB binder options were prepared in the road material laboratory of the Faculty of Civil Engineering, Czech Technical University in Prague where finely ground rubber of selected granularity was added to straight-run paving grade bitumen 50/70 or 70/100. The rubber was pulverized by the cooperating partner, company Lavaris s.r.o. The experimental designs of CRMB binders were mixed for at least 30 minutes under 170-180 °C with a laboratory homogenizer using a speed of 400-450 revolutions per minute. This mixing was done by IKA C-MAG HS7 magnetic stirrer for laboratory application.

Table 1. Assessed experimentally designed CRMB binder options

<table>
<thead>
<tr>
<th>Bituminous binder</th>
<th>Identification</th>
<th>Activated rubber powder</th>
<th>Content of the catalyst</th>
</tr>
</thead>
<tbody>
<tr>
<td>50/70</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>50/70 + 15% CR 0.3-0.5 + 10% of activating catalyst</td>
<td>15% CR_{0.3,0.5} +10% AK</td>
<td>0.3-0.5 mm</td>
<td>10 %</td>
</tr>
<tr>
<td>50/70 + 15% CR 0.3-0.5 + 2.5% of activ. catalyst</td>
<td>15% CR_{0.3,0.5} + 2.5% AK</td>
<td>0.3-0.5 mm</td>
<td>2.5 %</td>
</tr>
<tr>
<td>50/70 + 15% CR K1</td>
<td>15% CR_K</td>
<td>0.3-0.8 mm</td>
<td></td>
</tr>
<tr>
<td>50/70 + 15% CR K1 + 10% of activating catalyst</td>
<td>15% CR_K +10% AK</td>
<td>0.3-0.8 mm</td>
<td>10 %</td>
</tr>
<tr>
<td>50/70 + 15% CR K1 + 5% of a activating catalyst</td>
<td>15% CR_K +5% AK</td>
<td>0.3-0.8 mm</td>
<td>5 %</td>
</tr>
<tr>
<td>50/70 + 15% CR K1 + 2.5% of activating catalyst</td>
<td>15% CR_K + 2.5% AK</td>
<td>0.3-0.8 mm</td>
<td>2.5 %</td>
</tr>
<tr>
<td>50/70 + 10% CR K1 + 2.5% of activating catalyst</td>
<td>10% CR_K + 2.5% AK</td>
<td>0.3-0.8 mm</td>
<td>2.5 %</td>
</tr>
<tr>
<td>50/70 + 15% CR (ARP 0.0-0.8mm) + 10% of activating catalyst</td>
<td>15% CR_{0.0,0.8} + 10% AK</td>
<td>0.0-0.8 mm</td>
<td>10 %</td>
</tr>
<tr>
<td>50/70 + 15% CR (ARP 0.5-0.8mm) + 10% of activating catalyst</td>
<td>15% CR_{0.5,0.8} + 10% AK</td>
<td>0.5-0.8 mm</td>
<td>10 %</td>
</tr>
<tr>
<td>50/70 + 15% CR (ARP 0.0-0.8mm) with activating catalyst</td>
<td>15% ARP5AK_{0.0,0.8}</td>
<td>0.0-0.8 mm</td>
<td>5 %</td>
</tr>
<tr>
<td>50/70 + 10% CR (ARP 0.0-0.8mm) with activating catalyst</td>
<td>10% ARP5AK_{0.0,0.8}</td>
<td>0.0-0.8 mm</td>
<td>5 %</td>
</tr>
<tr>
<td>70/100 +15% CR K1 + 1% PPA</td>
<td>70/100 + 15% CR_K + PPA</td>
<td>0.3-0.8 mm</td>
<td>1 % PPA</td>
</tr>
</tbody>
</table>

As a basic bituminous binder typical paving grade bitumen 50/70 was selected. For this bitumen the harmonized European standard CSN EN 12591 specifies softening point range 46-54 °C and penetration range 50-70 dmm. From the perspective of the latter characteristic the used binder slightly exceeds the required range and it should rather be considered as a harder type of bitumen 70/100. Nevertheless, with respect to the resulting behavior of the bitumen in the pavement slightly higher penetration of 50/70 bitumen might have rather positive impact especially if the phenomenon of bitumen ageing is considered. Based on these basic criteria the approximate impact assessment of used combination with pulverized rubber (activated rubber powder – ARP) and selected chemical additive on the bitumen characteristics can be done. Comparison can be further done according to the requirements defined in the technical specifications of Ministry of Transportation of the Czech Republic (TP 148, 2009), for compacted asphalt mixtures with rubber modified bituminous binders. Simultaneously it is possible for sake of additional comparison use e.g. the requirements which are given in German technical specifications for standardized RmB (CRmB) binders (Technische Lieferbedingungen für Gummi- und Bitumen, 2010). In such a case the most suitable type seems to be the bitumen RmB 20/60-55. For these binders following parameters are defined (Table 2).
Table 2. Required values of German and Czech specifications for CRMB binders

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Unit</th>
<th>TP 148</th>
<th>RmB 20/60-55</th>
</tr>
</thead>
<tbody>
<tr>
<td>Penetration @25°C</td>
<td>dmm</td>
<td>25-75</td>
<td>20-60</td>
</tr>
<tr>
<td>Softening point (ring&amp;ball)</td>
<td>°C</td>
<td>&gt; 55</td>
<td>&gt; 55</td>
</tr>
<tr>
<td>Elastic recovery @25°C</td>
<td>%</td>
<td>-</td>
<td>&gt; 50</td>
</tr>
<tr>
<td>Complex shear modulus G* @ 60°C, 1.59 Hz, PP25, 2mm gap</td>
<td>Pa</td>
<td>-</td>
<td>≥ 7.000</td>
</tr>
<tr>
<td>Phase angle δ @ 60°C, 1.59 Hz, PP25, 2mm gap</td>
<td>°</td>
<td>-</td>
<td>&lt; 75</td>
</tr>
<tr>
<td>Dynamic viscosity @ 150 °C</td>
<td>Pa.s</td>
<td>0.5-1.0</td>
<td>-</td>
</tr>
</tbody>
</table>

4 TEST METHODS APPLIED

Standard empirical testing and selected functional characteristics test methods were used to assess the bitumen performance within the evaluation of the impact of the activated rubber powder in combination with new type of activating and stabilizing catalyst, which was added to the rubber-bitumen composite with the aim to get mainly a homogeneous and storage-stable product.

Table 3. Test methods applied

<table>
<thead>
<tr>
<th>Test method</th>
<th>Standard</th>
</tr>
</thead>
<tbody>
<tr>
<td>softening point determination by means of the ring and ball method</td>
<td>EN 1427</td>
</tr>
<tr>
<td>determination of needle penetration at 25 °C</td>
<td>EN 1426</td>
</tr>
<tr>
<td>determination of elastic recovery at 25 °C</td>
<td>EN 13398</td>
</tr>
<tr>
<td>storage stability test; 72±1 h and temperature of 180 °C</td>
<td>EN 13399</td>
</tr>
<tr>
<td>determination of the complex shear modulus G* and phase angle δ at 60 °C</td>
<td>EN 14770</td>
</tr>
<tr>
<td>multiple stress creep and recovery test (MSCR)</td>
<td>EN 16659</td>
</tr>
<tr>
<td>dynamic viscosity determination</td>
<td>EN 13302</td>
</tr>
</tbody>
</table>

5 SUMMARY OF TEST RESULTS

5.1 Empirical characteristics

Table 4 summarizes empirical test results. The required value of penetration for rubber-modified binders according to TP is prescribed by the range of 25 to 75 dmm. All versions of experimentally designed CRMB binders with various combinations of chemical catalyst quantity and with different particle size of the applied rubber fall within such range. If we compare the test results to the German specifications where the penetration range is narrower (20 to 60 dmm), the outcome remains unchanged.

The penetration values are rather well-balanced; the lowest values are achieved by the binder with the finest pulverized rubber and the lowest concentration of the activating catalyst at the same time. It was found partly strange, that the highest value was demonstrated by the binder with the same quantity of the chemical catalyst; however, 10 % of pulverized rubber of size 0.0-0.8 mm was applied. On the other hand, this is far from surprising if we take into account the fact that the presence of rubber in the binder always hinders penetration and such decrease increases with the quantity applied. From the point of view of the quantities of applied activating catalyst, no clear dependence has been observed.
In the case of the softening point, the presence of pulverized or mechanically ground rubber results in a higher value of this characteristic; in comparison to the input bitumen, the increase may amount up to 15 °C. No clear relationship between the quantity of the activating catalyst and the softening point has been observed in this case, either. To a certain point, this is reflected in the penetration index value; rubber with the same particle size, identical quantity and differing concentrations of the activating catalyst has no impact on the penetration index. The impact of activated pulverized rubber particle size on the softening point was observed with the application of 10 % activating catalyst. This is also reflected in the penetration index which clearly shows that the use of rubber with narrow fraction, 0.5-0.8 mm, with the same concentration of the activating catalyst, results in a slight decrease of penetration index which is linked to an expected slight worsening of bitumen thermal susceptibility. Generally, when compared to the input binder, the application of activated pulverized rubber shows improved penetration index in the sense of increased binder stiffness and, therefore improved resistance to deformation can be expected.

Table 4. Results of basic empirical bitumen tests

<table>
<thead>
<tr>
<th>Bituminous binder</th>
<th>Penetration point R&amp;B [0,1 mm]</th>
<th>Softening point R&amp;B [°C]</th>
<th>Penetration index [-]</th>
<th>Elastic recovery [%]</th>
<th>Storage stability [°C]</th>
</tr>
</thead>
<tbody>
<tr>
<td>50/70</td>
<td></td>
<td>71.2</td>
<td>47.5</td>
<td>-1.0</td>
<td>4.6</td>
</tr>
<tr>
<td>15% CR&lt;sub&gt;0.3-0.5&lt;/sub&gt; +10% AK</td>
<td>44.4</td>
<td>60.2</td>
<td>0.8</td>
<td>39.1</td>
<td>5.8</td>
</tr>
<tr>
<td>15% CR&lt;sub&gt;0.3-0.5&lt;/sub&gt; + 2.5% AK</td>
<td>36.3</td>
<td>61.7</td>
<td>0.6</td>
<td>33.6</td>
<td>-10.7</td>
</tr>
<tr>
<td>15% CR&lt;sub&gt;K&lt;/sub&gt;</td>
<td>41.0</td>
<td>63.1</td>
<td>1.1</td>
<td>35.1</td>
<td>-8.4</td>
</tr>
<tr>
<td>15% CR&lt;sub&gt;K&lt;/sub&gt; +10% AK</td>
<td>44.8</td>
<td>60.2</td>
<td>0.8</td>
<td>42.8</td>
<td>2.7</td>
</tr>
<tr>
<td>15% CR&lt;sub&gt;K&lt;/sub&gt; +5% AK</td>
<td>46.2</td>
<td>60.0</td>
<td>0.8</td>
<td>41.8</td>
<td>1.6</td>
</tr>
<tr>
<td>15% CR&lt;sub&gt;K&lt;/sub&gt; + 2.5% AK</td>
<td>42.2</td>
<td>62.3</td>
<td>1.1</td>
<td>46.0</td>
<td>-12.5</td>
</tr>
<tr>
<td>10% CR&lt;sub&gt;K&lt;/sub&gt; + 2.5% AK</td>
<td>49.6</td>
<td>55.7</td>
<td>0.1</td>
<td>31.0</td>
<td>-</td>
</tr>
<tr>
<td>15% CR&lt;sub&gt;0.0-0.8&lt;/sub&gt; +10% AK</td>
<td>43.0</td>
<td>61.5</td>
<td>0.9</td>
<td>47.6</td>
<td>-12.0</td>
</tr>
<tr>
<td>15% CR&lt;sub&gt;0.5-0.8&lt;/sub&gt; + 10% AK</td>
<td>42.4</td>
<td>58.9</td>
<td>0.4</td>
<td>39.6</td>
<td>-1.5</td>
</tr>
<tr>
<td>15% ARP5AK&lt;sub&gt;0.0-0.8&lt;/sub&gt;</td>
<td>40.2</td>
<td>63.6</td>
<td>1.2</td>
<td>48.6</td>
<td>0.2</td>
</tr>
<tr>
<td>10% ARP5AK&lt;sub&gt;0.0-0.8&lt;/sub&gt;</td>
<td>45.1</td>
<td>58.9</td>
<td>0.5</td>
<td>37.1</td>
<td>8.5</td>
</tr>
<tr>
<td>70/100 + 15% CR&lt;sub&gt;K&lt;/sub&gt; + PPA</td>
<td>40.0</td>
<td>60.9</td>
<td>0.7</td>
<td>38.6</td>
<td>2.7</td>
</tr>
</tbody>
</table>

In the assessment of elastic recovery of modified binders, we must primarily note that none of the CRMBs reached the 50 % minimum value adopted by the preliminary specifications, currently implemented in Germany. From the perspective of general findings, the value is in our opinion rather overestimated in the German specifications; it is usually only achievable if a small quantity of elastomers is also added to the bitumen-rubber composite. Obviously, the value is influenced more by the rubber granularity; none of the narrow rubber grading results in higher elastic recovery values; contrastingly, particle sizes from 0.0 mm to 0.8 mm come closest to the 50 % level. Analogously, no dependence has been clearly identified with the use of various concentrations of the activating catalyst. It is obvious that the application of the same type of crushed rubber and its identical concentration in the bitumen, the lowest elastic recovery is achieved by 5 % of catalyst applied. For the sake of comparison, the aforementioned binder with poly-phosphoric acid instead of the activating catalyst provides hints for thinking that the catalyst is likely to have a slightly better effect on the elastic behaviour of the CRMB composite.

From the perspective of producing the so-called ready-to-use binders (terminal blends), an unquestionably important characteristic is the storage stability of CRMB which at the same time constitutes a weakness of this type of binders. In this regard, several findings have been presented:

- The solution applying merely 2.5 % of activating catalyst does not work.
- The solution where activated rubber powder with a broader range of particle size distribution or with very finely pulverized rubber up to 0.5 mm does not work.

- On the contrary, the application of up to 10 % of activating catalyst seems to work very well; CRMB composite with the catalyst replaced by poly-phosphoric acid had very good results in this regard as well. The results so far suggest that versions of storage-stable binder do exist indeed.

- The comparison of a binder version with activating catalyst added during the pulverization process or separately during CRMB preparation presented no difference in the ultimate characteristics. It has only been demonstrated that the application of 5 % activating catalyst and a combination of such adjusted ground rubber with additives, 10 % of which is added to the bituminous binder, does not deliver a storage-stable (homogeneous) product.

5.2 Dynamic viscosity

The viscosity testing is mainly used for determining the ideal temperature range, either for storage, pumping, transport, mixing or asphalt mix compaction. The target temperature was specified to be the binder temperature at which the dynamic viscosity at $6.8 \text{ s}^{-1}$ (20 rpm) is $0.17 \pm 0.02 \text{ Pa.s}$ for mixing and $0.28 \pm 0.03 \text{ Pa.s}$ for compacting (Asphalt Institute, 1962). These values have been verified by in-situ practice and are a good fit for neat bituminous binders but have proven to be usually non-realistic for modified bitumen as they result in too high temperatures, causing binder degradation, excessive energy use and possibly dangerous volatile emissions. This might be valid for CRMBs as well. Activation energy, yield stress, and high- or low-shear rate viscosity testing methods were proposed to resolve this issue with the final finding that the dynamic viscosity temperatures resulting in the above mentioned viscosities should be reduced by 14-25 °C (Hensley, Parmer, 1998; Bahia et al. 2006).

Viscosity was determined both for specific temperatures and selected shear rate, and in the form of flow curves for broader thermal ranges. Dynamic viscosity tests conducted under 20 revolutions per minute (the speed considered by US technical specifications as a baseline value for the processing of bituminous binder in asphalt mixtures) presented best results, when assessing the CRMBs designed, for the option with 10 % of activated pulverized rubber. The result is more than logical.

If we focus solely on the options with 15 % pulverized rubber, it is obvious that the narrower granularity results in lower viscosity values (even in the case of activated rubber of particle size 0.5-0.8 mm). At the same time, it is obvious that higher applied concentrations of the activating catalyst reduce the viscosity value by roughly 14 % in comparison to binders without such catalyst. It is also noticeable that the addition of activating catalyst to finely pulverized rubber during the milling process, with subsequent application of the resulting material in bituminous binder, slightly increases the dynamic viscosity value. Contrastingly, we can note that the presence of PPA hasn’t any impact on dynamic viscosity; the viscosity values might deteriorate minimally from the point of view of conditions necessary for workability and pumping of hot bitumen. Under 150 °C and with the selected shear rate, none of the binders meets the requirements stipulated by TP 148. Nonetheless, the Technical Conditions do not require any specific shear rate.
The above shown results are also confirmed in cases where a different shear rate is selected, or where dynamic viscosity is monitored within a broader thermal range (100-150 °C) – Figures 2 and 3. Even this analysis records the lowest increase of the entire flow curve when 10 % pulverized rubber is applied in combination with the activating catalyst. Subsequently, the best binder is the one with the smallest particle sizes of mechanically activated rubber; basically, this is the expected trend. If we focus on the largest drop in dynamic viscosity depending on shear rate under the selected test temperature, such behaviour is most often
observed in bituminous binders with pure, activated finely pulverized rubber as well as in binder options where 2.5% activating catalyst was used in such rubber.

5.3 Multiple stress creep and recovery test (MSCR)

The multiple stress creep and recovery test of bituminous binders under various levels of stress applied, currently still a subject of intense expert debates, is presently considered the most appropriate method to verify the deformation characteristics of bituminous binders under high temperatures (Anderson & Bukowski 2011; FHWA 2011). The above applies primarily for polymer-modified bituminous binders (CRMB can be classified in this group, too, although it is not strictly a PMB). The key indicator is the irreversible shear compliance \( J_{nr} \) determined under the strain of 0.1 kPa and 3.2 kPa and at a selected temperature which, in the case of the bitumen assessments presented in this paper amounted to 60 °C.

The results show that, in compliance with standard findings, the value of \( J_{nr} \) increases with increasing test strain; the difference between modified and non-modified binders is the order of magnitude of the irreversible shear compliance, or elastic recovery achieved. This reflects the effect of improved elastic properties and, therefore, much better response to multiple stress. Generally, we can say that with increasing numbers of stress cycles, irreversible shear compliance increases and, contrastingly, the elastic recovery value follows an opposite trend. In general, it may be noted that from the perspective of deformation behaviour, the \( J_{nr} \) is required to be as low as possible while elastic recovery should demonstrate a higher value. In the context of this assumption, it can be noted that those options of bituminous binders perform the best which have no activating catalyst, or the proportion thereof does not exceed 2.5% catalyst by mass. It is obvious that the \( J_{nr} \) value increases with the growing quantity of catalyst. In the case of PPA, the effect is not as distinctive; the results basically correspond with the impact of 5% by mass of activating catalyst. It is also noticeable that the pulverized rubber quantity affects the irreversible shear compliance value and higher quantities result in improved deformation characteristics. In contrast to that, the granularity of mechanically activated pulverized rubber has no impact from the point of view of MSCR test. Last but not least, attention should also be paid to a comparison of bitumen options with the same type of mechanically activated pulverized rubber where the only difference is the timing of the activating catalyst addition during the high-speed milling or during CRMB production. The results show that adding the activating catalyst during the high-speed milling process partly improves the elastic recovery value as well as \( J_{nr} \). Table 5 summarizes the MSCR test results for both stress levels.

Table 5. MSCR test results for assessed CRMB binders

<table>
<thead>
<tr>
<th>Bituminous binder</th>
<th>Strain ε [%]</th>
<th>Elastic recovery [%]</th>
<th>( J_{nr} ) [kPa⁻¹]</th>
<th>Strain ε [%]</th>
<th>Elastic recovery [%]</th>
<th>( J_{nr} ) [kPa⁻¹]</th>
</tr>
</thead>
<tbody>
<tr>
<td>50/70</td>
<td>5.73</td>
<td>0.60</td>
<td>5.73</td>
<td>191.02</td>
<td>0.02</td>
<td>5.97</td>
</tr>
<tr>
<td>15% CR₀.₃,₀.₅ +10% AK</td>
<td>0.73</td>
<td>16.41</td>
<td>0.73</td>
<td>27.16</td>
<td>5.96</td>
<td>0.85</td>
</tr>
<tr>
<td>15% CR₀.₃,₀.₅ + 2.5% AK</td>
<td>0.54</td>
<td>17.01</td>
<td>0.54</td>
<td>19.61</td>
<td>7.18</td>
<td>0.61</td>
</tr>
<tr>
<td>15% CRK</td>
<td>0.39</td>
<td>34.83</td>
<td>0.39</td>
<td>19.10</td>
<td>9.46</td>
<td>0.60</td>
</tr>
<tr>
<td>15% CRK +10% AK</td>
<td>0.61</td>
<td>30.35</td>
<td>0.61</td>
<td>28.55</td>
<td>7.90</td>
<td>0.89</td>
</tr>
<tr>
<td>15% CRK +5% AK</td>
<td>0.47</td>
<td>33.53</td>
<td>0.47</td>
<td>23.03</td>
<td>9.22</td>
<td>0.72</td>
</tr>
<tr>
<td>15% CRK +2.5% AK</td>
<td>0.33</td>
<td>38.98</td>
<td>0.33</td>
<td>17.06</td>
<td>11.84</td>
<td>0.53</td>
</tr>
<tr>
<td>10% CRK +2.5% AK</td>
<td>0.82</td>
<td>28.03</td>
<td>0.82</td>
<td>43.28</td>
<td>3.27</td>
<td>1.35</td>
</tr>
<tr>
<td>15% CR₀.₀,₀.₈ + 10% AK</td>
<td>0.53</td>
<td>22.16</td>
<td>0.53</td>
<td>23.53</td>
<td>7.34</td>
<td>0.74</td>
</tr>
<tr>
<td>15% CR₀.₀,₀.₈ + 10% AK</td>
<td>0.57</td>
<td>28.11</td>
<td>0.57</td>
<td>26.01</td>
<td>7.22</td>
<td>0.81</td>
</tr>
<tr>
<td>15% ARP5AK₀.₀,₀.₈</td>
<td>0.29</td>
<td>22.57</td>
<td>0.29</td>
<td>19.17</td>
<td>10.53</td>
<td>0.60</td>
</tr>
<tr>
<td>10% ARP5AK₀.₀,₀.₈</td>
<td>0.68</td>
<td>28.5</td>
<td>0.68</td>
<td>33.18</td>
<td>5.73</td>
<td>1.04</td>
</tr>
<tr>
<td>70/100 + 15% CRK + PPA</td>
<td>0.47</td>
<td>31.80</td>
<td>0.47</td>
<td>23.01</td>
<td>8.43</td>
<td>0.72</td>
</tr>
</tbody>
</table>
5.3 Oscillation test (frequency sweep test) – determination of complex shear modulus

From the perspective of test set-up and, particularly, of overall evaluation thereof, dynamic oscillation measurements of bituminous binders can be divided into two approaches of the analyses performed most frequently. In the former case, the values of complex shear modulus were assessed for the typical frequency of 1.59 Hz and selected temperature which usually represent higher operation temperature levels. The results achieved are presented in Tables 6. The comparison indicates that:

- The quantity of activated pulverized rubber affects both the complex shear modulus value and the phase shift angle which indicates the degree of bituminous binder elasticity.
- The addition of activating catalyst with increasing proportion in the resulting CRMB reduces the complex shear modulus value – this shows a slightly negative impact on deformation characteristics.
- Better effects of narrower grading of pulverized rubber are noticeable; at the same time, from the perspective of higher stiffness values as expressed by the complex shear modulus, activated rubber with particle size of 0.3-0.5 mm appears to be more effective. In comparison to other results, activated pulverized rubber with particle size of 0.3-0.8 mm appears to be sufficiently well-balanced and demonstrates its advantages in other tests, too. The particle sizes of pulverized rubber should not be underestimated; on the other hand, if good storage stability is achieved, or if elastic recovery test values are fine, it is not necessary to use very narrow particle size grading only. In contrast to some tests mentioned previously, CRMB with added PPA demonstrates very good values of complex shear modulus and phase angle. The well-balanced character of the solution is indicated primarily by resistance to permanent deformation and resistance to fatigue parameters ($G^*/\sin(\delta)$; $G^*\sin(\delta)$) as defined in the past by the U.S. SHRP recommendations.

Table 6. Complex shear modulus and phase angle values for $T=60^\circ\text{C}$ and $f=1.59$ Hz

<table>
<thead>
<tr>
<th>Bituminous binder</th>
<th>$G'$ [kPa]</th>
<th>$G''$ [kPa]</th>
<th>$G^*$ [kPa]</th>
<th>$\delta$ [°]</th>
<th>$G^*/\sin(\delta)$ [kPa]</th>
<th>$G^*\sin(\delta)$ [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>50/70</td>
<td>0.2</td>
<td>3.0</td>
<td>3.0</td>
<td>86.5</td>
<td>3.0</td>
<td>3.0</td>
</tr>
<tr>
<td>15% CR$_{0.3-0.5}$+10% AK</td>
<td>4.1</td>
<td>12.1</td>
<td>12.8</td>
<td>71.5</td>
<td>13.4</td>
<td>12.1</td>
</tr>
<tr>
<td>15% CR$_{0.3-0.5}$+2.5% AK</td>
<td>5.3</td>
<td>14.8</td>
<td>15.7</td>
<td>70.3</td>
<td>16.6</td>
<td>14.7</td>
</tr>
<tr>
<td>15% CR$_K$</td>
<td>4.8</td>
<td>13.7</td>
<td>14.6</td>
<td>70.5</td>
<td>15.4</td>
<td>13.7</td>
</tr>
<tr>
<td>15% CR$_K$+10% AK</td>
<td>3.8</td>
<td>11.0</td>
<td>11.6</td>
<td>70.8</td>
<td>12.2</td>
<td>10.9</td>
</tr>
<tr>
<td>15% CR$_K$+5% AK</td>
<td>4.1</td>
<td>11.5</td>
<td>12.2</td>
<td>70.5</td>
<td>12.9</td>
<td>11.5</td>
</tr>
<tr>
<td>15% CR$_K$+2.5% AK</td>
<td>5.4</td>
<td>13.6</td>
<td>14.6</td>
<td>68.4</td>
<td>15.6</td>
<td>13.5</td>
</tr>
<tr>
<td>10% CR$_K$+2.5% AK</td>
<td>1.8</td>
<td>7.6</td>
<td>7.8</td>
<td>76.9</td>
<td>8.0</td>
<td>7.6</td>
</tr>
<tr>
<td>15% CR$_{0.0-0.8}$+10% AK</td>
<td>4.9</td>
<td>12.7</td>
<td>13.6</td>
<td>68.8</td>
<td>14.6</td>
<td>12.7</td>
</tr>
<tr>
<td>15% CR$_{0.0-0.8}$+10% AK</td>
<td>3.4</td>
<td>10.6</td>
<td>11.1</td>
<td>72.3</td>
<td>11.6</td>
<td>10.6</td>
</tr>
<tr>
<td>15% ARP5AK$_{0.0-0.8}$</td>
<td>3.1</td>
<td>8.7</td>
<td>9.2</td>
<td>70.6</td>
<td>9.8</td>
<td>8.7</td>
</tr>
<tr>
<td>10% ARP5AK$_{0.0-0.8}$</td>
<td>1.3</td>
<td>5.4</td>
<td>5.6</td>
<td>76.1</td>
<td>5.7</td>
<td>5.4</td>
</tr>
<tr>
<td>70/100 + 15% CR$_K$+PPA</td>
<td>4.7</td>
<td>13.0</td>
<td>13.9</td>
<td>70.2</td>
<td>14.7</td>
<td>13.0</td>
</tr>
</tbody>
</table>

Another option for analysing the data from the oscillation test and deformation behaviour of bituminous binders is testing in a defined temperature range of e.g. 20-60 °C with strain frequencies within the interval of 0.1-10 Hz covering different levels of traffic loading. Subsequently, the principle of superposition of temperature and time is applied and all values measured are related to a single reference temperature; in case of the results presented herein, this temperature is 20 °C. This allows proper and comprehensive assessment of the deformation effects of individual binders within really broad frequency ranges, thus permitting interpretation of various impacts of traffic loads and intensities affecting the pavement structure materials. The graphic expression used in this case is called master curve.
The master curve for the complex shear modulus allows reading the dependencies of this characteristic on stress frequencies representing various types and states of traffic loading. What might be rather interesting under the selected temperature (20 °C) are lower frequencies, approximately up to the 0.1 Hz level. With frequencies on the $10^{-4}$ Hz level, the differences between some designed and assessed CRMB versions are quite obvious; at the same time, the benefits of higher quantities of the activating catalyst are demonstrated not only by reaching higher values of the complex shear modulus but also with respect to the slightly lower value when the highest frequency is applied. This might indicate a lesser thermal susceptibility of the binder under comparable traffic load impact while the temperature is variable. The information might be significant from the point of view of assessing the effect of annual or even daily fluctuations in air temperature.
6 CONCLUSIONS

Based on the experimental analyses performed, we can note that the new type of activating catalyst has, if appropriate quantity of such additive is used in CRMB binder and if a suitable type of pulverized mechanically activated rubber is used, a positive effect on the resulting characteristics of the modified bitumen. This confirms the often-repeated finding that a higher proportion of activated rubber powder (ARP) in the bitumen improves the deformation characteristics of the material on one hand while, on the other hand, it deteriorates the dynamic viscosity value which, however, is an assumed dependence. Nevertheless, it was not confirmed that in this case and with this combination with the activating catalyst used it would have been impossible to achieve a homogeneous and, therefore, storage-stable product. Quite the contrary, some options of the CRMB binders demonstrated values where the differences in softening points after the storage stability tests did not exceed 3 °C. It was clearly shown that in comparison to a CRMB without the catalyst, the addition of a catalyst improves homogeneity, primarily in cases where 5-10 M% of the agent is added.

Logically, the particle size of pulverized rubber affects the resulting viscosity. We can notice that the application of activating catalyst reduces this value (cf. dynamic viscosity values under 150 °C in CRMB_15% CRK and CRMB_15% CR0.0-0.8 + 10%AK). The results of dynamic tests for determination of deformation behaviour prediction are not quite consistent. While, from the perspective of complex shear modulus, the options with 10 % of activating catalyst added either during rubber pulverization or CRMB preparation score very well, from the perspective of irreversible shear compliance very low values are achieved when just 15 % crushed rubber is used, or when the rubber and activator are mixed during crushing, or when only 2.5 % activating catalyst is applied.

Last but not least, we must mention that from the point of view of application of the activating catalyst as such, both solutions (either during rubber milling or CRMB production) are possible and the resulting binders reach similar scores. This indicates that the choice of 5 % activating catalyst is probably optimal or near-optimal from the perspective of technical and economic aspects. This means that with this quantity of additive, the characteristics of a CRMB-type binder are sufficiently retained while this solution has a slightly better economic scale if compared to the addition of e.g. 10 % activating catalyst. On the other hand, reducing the proportion of the catalyst to 2.5 % is not recommended. In such cases, it is better not to use any activating catalyst at all. From the perspective of the pulverized rubber grading, 0.0-0.8 mm or 0.5-0.8 mm rubber are preferred as the most suitable solutions; best effects are achieved in such cases even from the point of view of homogeneity of the resulting composite.

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Developing Abu Dhabi Sustainable Road Rating System (ADSRRS)

Greening Road Projects

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KEYWORDS:
roads, sustainability, rating system, scorecard, credits, Abu Dhabi.

ABSTRACT:
The Abu Dhabi Sustainable Road Rating System (ADSRRS) is a framework that evaluates and scores best practices in planning, design, construction, operation and maintenance of roads projects. It uses forty four (44) “criteria” (called “credits”) to award sustainable choices and practices in pre-defined areas of focus such as energy efficiency of lighting, urban heat, road safety, long live pavement, air pollution reduction, recycled materials, life cycle cost, optimized ROW, context sensitive solutions, pedestrian and bicycle facilities and sustainability in planting and irrigation. These credits (i.e. practices) are subjected to measurement and scoring, therefore are conceptually transferable to a rating system for roads using consistent terminology, approaches, thresholds, and formats. Road projects can earn points whenever sustainability credits are applied. The more credits applied in a road project, the higher score and award received (4 levels of awards). The purpose of this paper is to illustrate the new ADSRRS, how was developed, how it functions and what implementation challenges the system is facing.
Developing Abu Dhabi Sustainable Road Rating System (ADSSRS)

Dr. Jamal El Zarif
Khaled Al Junadi
Shamsa Al Muharrami

1 INTRODUCTION

1.1 What Is Sustainability?

There is no one definition of “Sustainability” that is universally accepted. However, almost all definitions recognize the goal of sustainability is to address the “triple bottom line”; the (1) economic (viability), (2) social (acceptability), and (3) environmental (friendliness) pillars. In other words, the goal of sustainability can be described as satisfying basic social and economic needs, both present and future, and the responsible use of natural resources, all while maintaining or improving the well-being of the environment on which life depends.

1.2 Why Sustainability for Road Projects?

Roadway projects have a great impact on cities vitality and resilience including the economy, built, health, social, and natural environments of their communities. Therefore, a sustainable urban road network system is envisioned as one that not only seeks to avoid harmful impact, but also seeks to support and serve as a foundation for a flourishing economy, a liveable community, and a healthy natural environment.

2 HOW ROAD PROJECTS ARE IMPLEMENTED IN ABU DHABI?

For many years the Municipality of Abu Dhabi City (ADM) has been practicing the development and construction of roads “as usual” using the Municipality design manuals that were based mainly on AASHTO standards. Consultants have been designing routinely the roads components including pavement, structures, and lighting, landscape, drainage and irrigation elements. In many cases the outcome was overdesigned road facilities, which means more land for wider Right-of-way, extra costs, more materials wasted, natural resources depleted and energy consumed, extended construction duration and unpleasant impact on community.

However, since 2009 the “as usual” practice started to change. ADM through its commitment to move towards more sustainable practices in building infrastructure, has launched several “green” initiatives that contributed to the wider Government’s vision towards the Abu Dhabi Sustainable Development, such as the Public Lighting Strategy, pavement recycling, green waste recycling, recycled concrete crashed aggregate, irrigation SCADA system and irrigation mater plan, etc., and the latest Abu Dhabi Sustainable Road Rating System (ADSSRS) developed by December 2015. Years earlier, The Abu Dhabi Urban Planning Council (UPC) developed a similar system called the “Estidama Pearls Community Rating System” for land and community development.

3 OBJECTIVES OF BUILDING THE RATING SYSTEM

The objectives of Abu Dhabi Department of Municipal Affairs in creating a sustainable roads rating system are to provide a system that:

- Demonstrates the value and achievement of Abu Dhabi road projects
- Incorporates sustainability early in the process of road projects development
- Incorporates Abu Dhabi sustainable practices, standards and regulations in the system
- Is flexible and can be applied in different contexts
- Supports interdisciplinary decision making
- Provides quantitative measures of sustainability features
- Is easy to handle and functions well for professionals
- Encourages stakeholder participation (agencies, public, advocates)
- Includes context sensitive solutions (CSS) as a basic element and ensure that the context is forward looking
4 DEVELOPMENT APPROACH OF ADSRRS

The first step towards the ADSRRS was in 2012 when Greenroads™ Rating System (which was developed in the US by the University of Washington – Seattle and CH2M HILL) was tested for the initial assessment of its compatibility against two projects in Abu Dhabi. The test projects were assessed and results were presented in a workshops held in May 2012. These projects were: 1) Al Salam Street Project; and 2) Samha and Rahba Roads Project. There were a total of 48 Greenroads™ credits reviewed against each of the two projects. The final outcome of this trial was a decision to build a rating system tailored to local conditions and requirements to serve Abu Dhabi Vision and its sustainable development objectives.

The actual journey of building the ADSRRS Version-1.0 started in mid-2014. The development of the system extended in sequential steps over a 14 months period. The various rating system characteristics and content were established through many meetings with staff within ADM divisions and other agencies.

4.1 Develop “baseline” version of ADSRRS

The building blocks of ADSRRS were Department of Municipal Affairs and Abu Dhabi sustainability initiatives, guidelines, standards and regulations, the Estidama Pearl Community Rating System, and the Greenroads™ Manual v 1.5. A ‘Baseline’ version of the Abu Dhabi sustainable roadways rating was developed. This system contained initially 66 individual credits.

4.2 Baseline version benchmarking

The “Baseline” version of the Abu Dhabi Sustainable Roads Rating System was benchmarked against eight other local and international rating systems such as Estidama PRRS, Estidama PCRS, Ireland NRA System, Australian IS Rating Scheme, CEEQUAL, FHWA INVEST, GreenroadsTM, and EnvisionTM. The results of the benchmarking process were tabulated in matrix form to show how the AD system topics compare with those in other systems, and perhaps identify other practices that DMAT may adopt to incorporate into the Pilot Version. Even though ADSRRS was benchmarked against other rating systems internationally, it was, however, developed and tailored upon sustainable roads policies, specifications and experience developed by Abu Dhabi Emirate local agencies over the past several years, and by incorporating guidance and input of local stakeholders (e.g. ADM, UPC, DoT, EAD, etc.).

4.3 Pilot version for Abu Dhabi Sustainable Roads Rating System

Based upon the results of benchmarking, the Pilot Version of Abu Dhabi Sustainable Roads Rating System was developed, including credit documentation. The credit documentation includes a description for each credit, along with the goal, performance requirements, and references to guidelines, standards, or codes.

4.4 Verification tests against sample of projects

On completion of development phase, the completed Pilot Version of ADSRRS was reviewed against eight (8) existing recently developed road projects of different types including Small Urban, Small Rural, Major Urban and Major Rural Road Types with cost projects ranging from AED 4.5 to 165 Million UAE dirhams. Appendix A exhibits a summary of the outcome of these projects tested against the Pilot Version. The summary of these reviews acts as a means to confirm both the applicability and the benefits of the Pilot system and provide an indication for how it could be implemented in practice. The outcome of stage was a revised version (named Version-1.0) of the ADSRRS confirmed and documented.

4.5 Analysis of category coverage

The eight benchmarked rating systems addressed in Section 4.2 include 107 different credits in 7 sustainability topic categories. Most credit topics appear in multiple rating systems and a few appear in all of the rating systems. The ADSRRS is proposed to have 44 credits. Table 1 summarizes the number of credits proposed in the ADSRRS by the same Sustainability Topic Categories (excluding Innovation). The ADSRRS credits represent an even distribution of credits across the sustainability topic categories, and are also consistent with the total number of credit topics appearing in each category topic except Operation and Maintenance category, which ADSRRS has a major emphasis on this topic.
Table 1. Proposed ADSRRS credits by sustainability topic category

<table>
<thead>
<tr>
<th>Sustainability Topic Category</th>
<th>Number of Credits in Sustainability Topic Category for 8 Benchmarked Rating Systems</th>
<th>Number of Credits Proposed for ADSRRS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Planning</td>
<td>22</td>
<td>12</td>
</tr>
<tr>
<td>Public Good</td>
<td>17</td>
<td>8</td>
</tr>
<tr>
<td>Materials</td>
<td>14</td>
<td>4</td>
</tr>
<tr>
<td>Energy Conservation</td>
<td>11</td>
<td>3</td>
</tr>
<tr>
<td>Ecological Conservation</td>
<td>19</td>
<td>5</td>
</tr>
<tr>
<td>Construction</td>
<td>21</td>
<td>7</td>
</tr>
<tr>
<td>Operations &amp; Maintenance</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>Innovation</td>
<td>N/A</td>
<td>1</td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td><strong>107</strong></td>
<td><strong>44</strong></td>
</tr>
</tbody>
</table>

5 SYSTEM CHARACTERISTICS

The term “system characteristics” is intended to refer to a range of system design features that define the sustainability performance tool and how it will be used. Table 2 System Characteristics for ADSRRS defines the system characteristics and identifies the recommendation for ADSRRS.

Table 2. System characteristics for ADSRRS

<table>
<thead>
<tr>
<th>System Characteristic</th>
<th>Definition</th>
<th>Recommendation for ADSRRS</th>
</tr>
</thead>
<tbody>
<tr>
<td>System Owner</td>
<td>Who will own, maintain, and update the rating system?</td>
<td>Initially Abu Dhabi Department of Municipal Affairs and Transport</td>
</tr>
<tr>
<td>Transport or Infrastructure Focus</td>
<td>Will the system be focused on transport projects, or be part of an evaluation for civil infrastructure?</td>
<td>Transportation projects only</td>
</tr>
<tr>
<td>Type of System</td>
<td>Will this be a self-evaluation system or require agency or 3rd party certification? Is it voluntary or mandated?</td>
<td>Voluntary, self-evaluating first for a transition period, then Mandatory with Agency Certification.</td>
</tr>
<tr>
<td>Phase of Project Lifecycle</td>
<td>What point in the project life cycle should the initial system apply?</td>
<td>ADSRRS focuses on project development phase, with credits designed to enable sustainability enhancements during the entire project life cycle including the operations and maintenance phase for projects.</td>
</tr>
<tr>
<td>Relationship to Existing Regulations</td>
<td>Should the tool be all “above and beyond” or should it also reiterate sustainability guidelines</td>
<td>Combination - Reiterates standards that address sustainability (required credits) and has “above and beyond” (voluntary) credits</td>
</tr>
<tr>
<td>Credit Organization</td>
<td>Should the credits be organized by phase or topic?</td>
<td>Phase and Topic</td>
</tr>
<tr>
<td>Weighting of Credits</td>
<td>Should credits be weighted? If so, how?</td>
<td>Yes, by duration and magnitude of sustainability benefit</td>
</tr>
<tr>
<td>Pre-Requisites</td>
<td>Is it necessary to achieve pre-requisites in order to use the system?</td>
<td>Yes. There will be a set or required credits that reiterate already mandatory standards.</td>
</tr>
<tr>
<td>Promotion of Innovation</td>
<td>Should extra credits be allowed to promote innovation?</td>
<td>Yes, there will be a Voluntary “Innovation” credit or credits in a way that rewards the result</td>
</tr>
<tr>
<td>Differentiation by Project Type</td>
<td>Should the system fit all projects and allow differentiation by project type?</td>
<td>Yes, allow differentiation using scorecards that combine appropriate credits. For example, urban projects versus non-urban projects or small versus large projects.</td>
</tr>
</tbody>
</table>
6 ADSRRS WEIGHTING OF CREDITS

ADSRRS is designed as a weighted system. The overall goal of weighting is to make the point value for each credit commensurate with its potential to affect sustainability both in terms of significance and duration of the impact. Points are given to credits based on the following:

1. The span of impact to sustainability principles (environmental, economic, social, and cultural)
2. The relative magnitude of impact to sustainability (low, medium, high)
3. The relative duration of the impact to sustainability (upon construction, short term, long term).
4. To provide emphasis to selected regional and national priorities, such as water conservation.

The sum of points for each span, duration, magnitude of impact, and priority were then multiplied together to come up with the raw points for each credit. This raw point number was then normalized to provide points between 1 and 15 points per credit shown in Table 4. Examples showing how the scaled points of credits were calculated are shown in Appendix B. A comparative review of weighting schemes used for other rating systems is provided below in Table 3. The systems give progressive points for some or all of the credits in the system, where partial points are awarded for achieving a portion of the credit, and then additional points are awarded for achieving all aspects of the credit.

<table>
<thead>
<tr>
<th>Rating System</th>
<th>Point Range per Credit</th>
<th>One Number per Credit or Progressive</th>
<th>System has Required Credits? Or all Voluntary?</th>
<th>Overall Point total for Voluntary Credits</th>
<th>Basis of Points and Weights</th>
</tr>
</thead>
<tbody>
<tr>
<td>US FHWA INVEST</td>
<td>1-to-10</td>
<td>Progressive</td>
<td>All Voluntary</td>
<td>126</td>
<td>Agency Decided</td>
</tr>
<tr>
<td>GF Greenroads</td>
<td>1-to-5</td>
<td>Progressive</td>
<td>Required &amp; Voluntary</td>
<td>108</td>
<td>Academic</td>
</tr>
<tr>
<td>CEEQUAL V4</td>
<td>1-to-23</td>
<td>Progressive</td>
<td>All Voluntary</td>
<td>Varies Based upon Scoping</td>
<td>Stakeholder Input</td>
</tr>
<tr>
<td>ASCE ENVISION V2</td>
<td>1-to-25</td>
<td>Progressive</td>
<td>All Voluntary</td>
<td>Varies Based upon Scoping</td>
<td>Stakeholder Input</td>
</tr>
<tr>
<td>Estidama Communities</td>
<td>1-to-14</td>
<td>Progressive</td>
<td>Required &amp; Voluntary</td>
<td>159</td>
<td>Agency Decided</td>
</tr>
<tr>
<td>Estidama Public Realm</td>
<td>1</td>
<td>One Number</td>
<td>Required &amp; Voluntary</td>
<td>20</td>
<td>Agency Decided</td>
</tr>
<tr>
<td>ADSRRS Pilot Version</td>
<td>1-to-15</td>
<td>Progressive</td>
<td>Required &amp; Voluntary</td>
<td>220</td>
<td>Weighting Process</td>
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</table>
Table 4. ADSRRS credits and weight by project type and topic category

<table>
<thead>
<tr>
<th>Credit ID</th>
<th>Recommended Credit for AD SRRS V1.0</th>
<th>Scaled Points (1 to15)</th>
<th>Road Rehab min AED 500,000</th>
<th>Small Rural AED 2,000,000 –75,000,000</th>
<th>Small Urban ≥ AED 75,000,000</th>
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</thead>
<tbody>
<tr>
<td>PP-01</td>
<td>Integrated Development Strategy</td>
<td>15</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>PP-02</td>
<td>Lifecycle Cost Analysis</td>
<td>6</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>PP-03</td>
<td>Lifecycle Inventory</td>
<td>6</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>PP-04</td>
<td>Natural Systems Design &amp; Management</td>
<td>6</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>PP-05</td>
<td>Plan 2030</td>
<td>10</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>PP-06</td>
<td>Urban Systems Assessment</td>
<td>3</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>PP-07</td>
<td>Community Water Strategy</td>
<td>8</td>
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<td>●</td>
<td>●</td>
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<td>PP-08</td>
<td>Environmental Review Process</td>
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<td>●</td>
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<td>PP-09</td>
<td>Neighborhood Connectivity</td>
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<td>●</td>
<td>●</td>
<td>●</td>
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<td>PP-10</td>
<td>Guest Worker Accommodation</td>
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<td>●</td>
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<td>PP-11</td>
<td>Open Space Network</td>
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<td>●</td>
<td>●</td>
<td>●</td>
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<td>●</td>
<td>●</td>
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<tr>
<td><strong>Design</strong></td>
<td>[Public Good (PG), Materials (MT), Energy Conservation (EN) and Ecological Conservation (EC)]</td>
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<td></td>
<td></td>
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<tr>
<td>PG-01</td>
<td>Sustainability Educational Outreach</td>
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<td>●</td>
<td>●</td>
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<td>PG-02</td>
<td>Context Sensitive Solutions</td>
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<td>●</td>
<td>●</td>
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<td>PG-03</td>
<td>Cultural &amp; Historic Preservation</td>
<td>5</td>
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<td>●</td>
<td>●</td>
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<td>PG-04</td>
<td>Safety</td>
<td>11</td>
<td>●</td>
<td>●</td>
<td>●</td>
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<tr>
<td>PG-05</td>
<td>Pedestrian Access</td>
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<td>●</td>
<td>●</td>
<td>●</td>
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<tr>
<td>PG-06</td>
<td>Bicycle Access</td>
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<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>PG-07</td>
<td>Transit &amp; HOV Access</td>
<td>8</td>
<td>●</td>
<td>●</td>
<td>●</td>
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<tr>
<td>PG-08</td>
<td>Traffic Emissions Reduction</td>
<td>6</td>
<td>●</td>
<td>●</td>
<td>●</td>
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<tr>
<td>MT-01</td>
<td>Reuse/Recycled Materials</td>
<td>8</td>
<td>●</td>
<td>●</td>
<td>●</td>
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<tr>
<td>MT-02</td>
<td>Regional Materials</td>
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<td>MT-03</td>
<td>Long Life Pavement</td>
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<td>MT-04</td>
<td>Introduction of Innovative Design Materials</td>
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<td>●</td>
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<td>EN-01</td>
<td>Smart Infrastructure Systems</td>
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<td>●</td>
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<td>EN-02</td>
<td>Energy Efficiency Lighting</td>
<td>8</td>
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<td>EN-03</td>
<td>Urban Heat Reduction</td>
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<td>EC-01</td>
<td>Sustainability in Stormwater Management</td>
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<td>EC-02</td>
<td>Sustainability for Plantings &amp; Irrigation</td>
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<td>●</td>
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<td>EC-03</td>
<td>Ecological Connectivity</td>
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<td>CN-02</td>
<td>Construction Quality Control Plan</td>
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<td>●</td>
<td>●</td>
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<td>CN-03</td>
<td>Construction Environmental Management</td>
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<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>CN-04</td>
<td>Construction Environmental, Health &amp; Safety</td>
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<td>●</td>
<td>●</td>
<td>●</td>
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<td>CN-05</td>
<td>Construction Air Pollution Reduction</td>
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<td>●</td>
<td>●</td>
<td>●</td>
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<td>CN-06</td>
<td>Construction Site Recycling Plan</td>
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<td>CN-07</td>
<td>Construction Contractor Warranty</td>
<td>3</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td><strong>Operations &amp; Maintenance (OM) &amp; Innovative Practice (IP)</strong></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>OM-01</td>
<td>Facilities Maintenance Plan</td>
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<td>●</td>
<td>●</td>
<td>●</td>
</tr>
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<td>OM-02</td>
<td>Pavement Management System</td>
<td>5</td>
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<td>●</td>
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<tr>
<td>OM-03</td>
<td>Maintenance Management System</td>
<td>5</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>OM-04</td>
<td>Bridge Management Systems</td>
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<td>●</td>
<td>●</td>
<td>●</td>
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<tr>
<td>IP-01</td>
<td>Innovating Practice</td>
<td>8</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
</tbody>
</table>

| Total Number of Points and Credits | 293 | 17 | 22 | 39 | 32 | 43 |
7 HOW THE SYSTEM WORKS?

ADSRRS will be implemented as a voluntary sustainability rating system for roadway planning, design, construction and operation stages. It is applicable to all types and sizes of roadway projects, including new, rehabilitation, reconstruction, preservation, including all road components such as structures (bridges and tunnels), walkways, drainage, lighting, irrigation and landscape, etc.

It is envisioned there are minimum required credits called “Project Requirements” and the rest optional credits called “Voluntary Credits”. Achieving Voluntary Credits can earn points toward a total score for the project. To be a certified project as per ADSRRS, the project needs to meet all of the Project Requirements that represent about 25% of the total point score. Higher level of award needs to achieve a further threshold of at least 15% of total maximum points available by voluntary credits score. There are three higher levels of ADSRRS ratings based on how many points are earned within the Voluntary Credits.

7.1 Definition of project types and credits by type

The ADSRRS proposes five scorecards that pre-scope or identify credits that are most likely to be applicable for the project type. Selecting the proper scorecard for the project will help ensure that only relevant credits are evaluated against the specific project. The five scorecards are briefly described below.

1. **Road Rehabilitation**: for projects that are devoted exclusively to pavement preservation or restoration projects that extend the service life of existing facilities, ride quality and enhance safety. Minimum project budget is 500,000 AED.

2. **Small Rural**: for small rural reconstruction or rural structure replacement project, located outside Urban Boundaries; within budget range of 3,000,000-to-75,000,000 AED.

3. **Small Urban**: for small urban reconstruction, structure replacement project, streets and access lanes would be part of Small Urban; within budget range of 3,000,000-to-75,000,000 AED.

4. **Major Rural**: for rural construction project of a new roadway facility or a structure where nothing of its type currently exists and major reconstruction projects, located outside of Urban Boundaries; budget greater than 75,000,000 AED.

5. **Major Urban**: for urban construction project of a new roadway facility or a structure where nothing of its type currently exists and major reconstruction projects. Highways, Boulevards, Arterials would be part of Major Urban unless they are only repaving projects on those roads, located within Urban Boundaries; budget greater than 75,000,000 AED.

Projects to be excluded from mandatory application of the ADSRRS would include:

- Projects at or beyond 90% percent complete (design phase)
- Very small projects with budgets less than 500,000 AED (Construction Cost)
- Projects that construct only baseline infrastructure (utilities)

Based upon the pre-scoped project types, the proposed credits that must be considered for each project type are listed in detail in Table 4. This list was established based upon the findings in the verification tests of projects. A summary of the number of credits applicable to each project type is as follows: Road Rehabilitation – 17 credits, Small Rural – 22 credits, Small Urban – 34 credits, Major Rural – 32 credits, Major Urban – 43 credits.
7.2 ADSRRS proposed thresholds

The total number of points a project can earn, for each scorecard, can be used to develop thresholds, or achievement levels, that serve as relative benchmarks for sustainability accomplishments. Four scoring thresholds are identified. The proposed rating levels are based on points achieved out of total possible points. The lowest threshold is based upon achieving all of the 25% (required) credits and would be worth one “pearl”. Then the next threshold is 40% of points, which consists of 73 required points plus 44 voluntary credit points. Then additional 20% of points achieve a higher threshold that would be worth three “pearls”. The highest threshold equals 75% of all points for four “pearls” (see Table 5).

Achievement levels are likely to be revised over time as the Version 1.0 gets further tested and evaluated on projects to ensure that they are both practical and consistent.

<table>
<thead>
<tr>
<th>Credit and Credit Points Earned</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>73 Required Points (25%)</td>
<td>1</td>
</tr>
<tr>
<td>73 Required Points + 44 Voluntary Credit Points (40%)</td>
<td>2</td>
</tr>
<tr>
<td>73 Required Points + 103 Voluntary Credit Points (60%)</td>
<td>3</td>
</tr>
<tr>
<td>73 Required Points + 147 Voluntary Credit Points (75%)</td>
<td>4</td>
</tr>
</tbody>
</table>

8 ANTICIPATED BENEFITS OF ADSRRS

Roadway projects can have both direct and indirect impacts in numerous ways on the long term sustainability of the social, economic, and natural environments on which healthy communities depend. The development of SRRS can help to integrate triple bottom line thinking into the project development process and include the anticipated benefits described below:

- Define basic sustainability attributes for Abu Dhabi roadways
- Provide an improved evaluation framework of roadway project tradeoffs and decisions
- Encourage greater participation in roadway sustainability
- Promote interoperability with other applied guidelines and initiatives
- Allow for innovation with a system that is outcome-oriented
- Stimulate the market for sustainable practices and products
- Confer recognition for achievements on sustainable projects
- Save money over the project or program life cycle

Those tangible and non-tangible benefits including quantifiable financial and economic returns for short term (before-after analysis) and long term (with vs. without analysis) horizons are better assessed once the system is implemented and evaluated.

9 IMPLEMENTATION CHALLENGES OF THE SYSTEM

In December 2015, The Executive Council of Abu Dhabi Emirate advised to establish a Higher Steering Team from 9 relevant government agencies to prepare launching the ADSRRS and following-up implementation.

9.1 Transition from voluntary use to mandatory use of ADSRRS

The initial version of ADSRRS will be voluntary use. This will allow the DMAT to have users learn how to use the system and to potentially make updates before making the system mandatory. Preparations for making the ADSRRS mandatory use should include many of the next steps actions:
9.2 Certifications by all phases in project life cycle

The ADSRRS has identified actions that apply mostly during design and construction phases. Confirmation that these actions have been taken would occur after full completion of the project has been accomplished. The confirmation for operations & maintenance could be a certification after 2 years, or 5 years after construction completion.

9.3 Identify and decide on pre-requisite credits

The initial ADSRRS was established to have all credits be voluntary use. Eventually it will be important to transition so that some credits become required or pre-requisite credits. The decision of transition will become apparent over time as the system is tested.

9.4 Decide on ultimate threshold levels award titles

The scoring and weighting would need to be changed at one point of time in the future. Also the threshold levels must then be adjusted. Another continuing issue to manage will be the weighting of the credits. If certain credits become required, that adjustments to the weighting of credits and points will be necessary.

9.5 Prepare for certification process

Transitioning to a certification process will require the following key steps:

• Clarify the start date for application of projects certification.
• Users of the rating system and reviewers of submissions must be trained and certified.
• A means must be established for formal submissions of proofs of accomplishment of the credit requirements.

9.6 Develop obligatory conditions for contracting

The DMAT may choose to make the credit specifications required as part of design, construction, and operations & maintenance contracts. Preparations can be made to integrate the requirements of the credits into the scopes of work and contracts requirements for future contracting.

9.7 Long term plans for ADSRRS system ownership

A decision must be made regarding the ultimate owner and maintenance of the rating system. Initially the DMAT will be maintaining the system. The UPC is developing the Estidama Public Realm Rating System (PRRS) that has new credits for the non-roadway portions of the road cross section. Planning for future merger between the two systems under a joint operation, or under one owner agency can be conducted.

9.8 Further communications and training

The DMAT should continue to provide communications regarding the current version and changes to the rating system. These communications can be provided in several ways such as the website, orientation presentations to stakeholders, training sessions, demonstration projects and documentation of findings in case study reports to application of the ADSRRS.

9.9 Means to receive, manage, and respond to comments

DMAT staff should have a means to receive, manage, and respond to comments and potential ideas for future updates to the ADSRRS v1.0. These comments and inputs can be used for identifying potential updates for the next versions of ADSRRS.

9.10 Management and staffing for future ADSRRS operations

All sustainability rating systems will need to have staff support to manage and operate the rating system. Staffing levels required will depend upon the pace of implementation and operation.

(End of Paper)
### Appendix A. Summary results of projects tested against the Pilot Version of ADSRRS

<table>
<thead>
<tr>
<th>No</th>
<th>Project Title</th>
<th>Type</th>
<th>Case</th>
<th>Score</th>
<th>PP</th>
<th>PG</th>
<th>MT</th>
<th>EN</th>
<th>EC</th>
<th>CN</th>
<th>OM</th>
<th>IP</th>
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</thead>
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<tr>
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<td>RAW</td>
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**PP** = Project Planning  
**PG** = Public Good  
**MT** = Materials  
**EN** = Energy Conservation  
**EC** = Ecological Conservation  
**CON** = Construction  
**OM** = Operation and Maintenance  
**IP** = Innovating Practice

The **Raw (RAW)** score actually achieved by the project.

The **Potential (POT)** score that the project could have achieved without a substantial increase or change in project scope or cost.

The **Maximum Feasible (M.F)** score that the project could have achieved. This represents the sum of points that would have been feasible to apply on the project if the scope were increased and sufficient funds were available.
Appendix B. Sample of credits scale points calculation

<table>
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<th>Sustainable Credit</th>
<th>Scoring – Span of Impact</th>
<th>Scoring – Duration of Impact*</th>
<th>Scoring – Magnitude of Impact</th>
<th>Pilot Scoring**</th>
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Notes:

* Duration of Impact
- Short-term = the first 3-5 years of operation
- Long-term = more than 5 years of operation

** Pilot Scoring:
- Maximum RAW Points observed was 24, which was scaled to 15,
- At this stage, it has been assumed for this version that all credits are equally important (priority =1)
<table>
<thead>
<tr>
<th>PAPER TITLE</th>
<th>CARBON EMISSIONS FROM ROADS: MODEL DEVELOPMENT AND CASE STUDIES</th>
</tr>
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<tr>
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<th>5.2: GREENING ROAD PROJECTS</th>
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<td>Rezaul CHOWDHURY</td>
<td>Associate Professor</td>
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<td>United Arab Emirates</td>
</tr>
<tr>
<td>Munjed MARAQA</td>
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<td>Assistant Professor</td>
<td>Department of Civil and Environmental Engineering United Arab Emirates University</td>
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<tr>
<td>Qasim KHAN</td>
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<tr>
<th>E-MAIL (for correspondence)</th>
<th><a href="mailto:rezaulkabir@uaeu.ac.ae">rezaulkabir@uaeu.ac.ae</a></th>
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ABSTRACT:

While many countries have developed models to quantify GHG emissions from roads, none is available in the United Arab Emirates (UAE). The available commercial softwares are not comprehensive enough to quantify GHG emissions from the full life cycle of roads. This study aims to develop a model (referred to as RoadCO₂ model) that allows quantification of the amounts of CO₂ and other GHGs during the full life cycle of roads. The model divided the road life cycle into four stages: (a) pre-construction, (b) construction, (c) operation, and (d) rehabilitation and maintenance. The model is useful for scenario and policy analyses intended to make the roads more sustainable towards the environment. The model was used to estimate carbon emissions associated with road construction and operation for three cases in Abu Dhabi Emirate. Since national emission factors have not yet been established in the UAE, the IPCC emission factors were considered in the model. The three cases were part of Al Salam street upgrading, Al Rahba city internal road network, and widening of the Eastern Corniche road projects. The total emissions from these roads ranged from 541 to 11568 tons CO₂ eq/km/lane during their construction phase. The equipment used in their construction contributed about 70%, 15% and 20% of the total construction phase emissions, respectively. The rest of the emissions originated from the construction materials. Upgrading of Al Salam Street to an expressway and construction of the tunnel project produced the highest emissions from construction materials. Annual total emissions from traffic volumes in Al Salam Street segment was estimated to be over 37763 tons CO₂ eq, whereas emissions from traffic operations in Al Rahba city internal road segment was found to be 2028 tons CO₂ eq. Heavy and light vehicles dominate in Al Salam Street, whereas passenger cars dominate in Al Rahba city internal road network. Emissions due to utilization of electricity for lighting, traffic signals and for irrigation were not included in this study.
1 INTRODUCTION

Energy consumption and CO$_2$ emission rates in the world are increasing dramatically. Climate change has emerged as an important threat to economic development, environment, and public health. Greenhouse gases (GHGs) are the gases that trap heat in our atmosphere, creating what we call the “greenhouse effect”, which leads to warming the planet Earth. Human activities are contributing several types of GHGs to the atmosphere. Growing human population creates broad and collective impacts. This challenge is even more difficult in rapidly urbanizing regions, where the pace of change should be considered in engineering designs, modeling, planning, and policy development (White et al., 2010). The six GHGs identified by the Kyoto Protocol are carbon dioxide (CO$_2$), methane (CH$_4$), nitrous oxide (N$_2$O), hydrofluorocarbons (HFC), perfluorinated compounds (PFC), and sulfur hexafluoride (SF$_6$) (Galli et al., 2012). Each one of these gases can stay in the atmosphere for different amounts of time that can range from a few years to thousands of years (Chowdhury et al., 2016). While a lot of policies analysis has focused on CO$_2$ emissions from burning fossil fuels, which comprise about 60% of the total global GHG emissions in 2010, the United Nations Framework Convention on Climate Change (UNFCCC) and the Kyoto Protocol covered a wider array of CO$_2$ sources and of warming substances including the six gases mentioned earlier (IPCC, 2014).

Road transport has grown continuously over the last decades and further increase in the demand for transport is projected (Smit et al., 2008). Nonetheless, the transportation sector is one of the major contributors to global climate change through emissions of CO$_2$ and other GHGs. The transportation sector GHG emissions mainly involve fossil fuels burned for road, rail, air, and marine transportation. Transportation sector was responsible for about 7.0 GtCO$_2$eq of direct GHG emissions (including non-CO$_2$ gases) globally. It was responsible for approximately 23% of the total energy-related CO$_2$ emissions (IPCC, 2014). Roads contributed about 72% of the total direct and indirect GHGs emissions of the transportation sector. This means that roads are responsible for about 5.116 GtCO$_2$eq of direct and indirect emissions, ranking them as the top contributor of emissions within the transportation sector (IPCC, 2014). Roads contribute high amounts of emissions to the atmosphere due to their characteristics like the high-energy consumption, the use of resources such as raw material and land surface, the generation of high volumes of waste, and the quantity of linked transports and long service life (Barandica et al., 2013).

Abu Dhabi Emirate is the largest Emirate in the United Arab Emirates (UAE), occupying about 87% of the whole country. The total paved road network in the UAE is about 4080 km, ranking it the 156th in the world (CIA, 2017). The total length of external roads in Abu Dhabi Emirate is about 2705 km (SCAD, 2015). The transportation sector in Abu Dhabi Emirate contributed about 18.547 MtCO$_2$eq to the atmosphere in 2010. CO$_2$ dominated these emissions at 98.78%, while the remaining 1.22% consisted of the other GHGs (EAD, 2012). Roads accounted for about 63% of the total direct GHG emissions in the transport sector; this is mainly due to the extensive and well developed road network in the emirate.

Carbon footprint is a common term used to describe the total amount of CO$_2$ and other GHG emissions for which an individual or organization is responsible (ADB, 2010). The carbon footprint method is a tool that is capable of informing decision on resource and process selection for better understanding. It is also used to measure and reduce the impact of a system on the environment (Spray et al., 2014). The objective of this study was to develop a carbon footprint estimation model for road projects. Another objective was to utilize the model to estimate carbon footprint emissions associated with road construction and operation in the Abu Dhabi Emirate. For this purpose, a model (referred to as RoadCO$_2$) was developed after an in depth research, analysis, and data gathering. This model was created based on a huge database that contains many factors contributing to GHG emissions of the road projects over its respective life cycle. The model is based on the Intergovernmental Panel on Climate Change (IPCC) tier 1 due to the lack of a detailed estimation model and UAE specific emission factors. Nevertheless, the RoadCO$_2$ model can (and
should) be updated with local data to ensure precise results. This model was used to estimate the carbon footprint of three road projects within the Emirate of Abu Dhabi.

2 METHODOLGY

2.1 RoadCO₂ MODEL DEVELOPMENT

The framework of the developed model is schematically shown in Figure 1. The model accounts for all phases involved in a road project including pre-construction, construction, operation, and maintenance and rehabilitation. The activities involved in each phase are categorized into different groups as listed in Table 1. For example, the categories of the construction phase include sewerage works, water network works, road works, stormwater network works, telecommunication works, irrigation works, landscaping and street furnishing works, and lightening and electrical works. Activities for the construction phase are taken from the items listed in the Bill of Quantity (BOQ). This would be also applicable to the maintenance and rehabilitation phase as it involves the same activities that could be encountered during road construction. Resources needed to carry on any activity fall into one or more of three aspects: transportation, material, and equipment. Resources are quantified for the activity and quantified resources are multiplied by the appropriate emission factors to estimate the GHG emissions.

![RoadCO₂ Model](image)

Figure 1. Framework of the developed RoadCO₂ model
Table 1. Possible items included in the categories, activities and resources within each phase of the RoadCO₂ model

<table>
<thead>
<tr>
<th>Phase</th>
<th>Category</th>
<th>Activity</th>
<th>Resources</th>
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<tbody>
<tr>
<td>Pre-construction</td>
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<td>- Field tests</td>
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<tr>
<td></td>
<td>Site visits and testing</td>
<td>Laboratory tests</td>
<td>- Laboratory tests</td>
</tr>
<tr>
<td></td>
<td>Site preparation</td>
<td>Water diversion</td>
<td>- Water diversion</td>
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<tr>
<td></td>
<td></td>
<td>Cleaning a grubbing</td>
<td>- Cleaning a grubbing</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Removal of existing utilities</td>
<td>- Removal of existing utilities</td>
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<td></td>
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<td>Protection of existing</td>
<td>- Protection of existing</td>
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<td>Transportation</td>
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<td>- Transportation</td>
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<td>Rehabilitation</td>
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<td>- Equipment</td>
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<td>Stormwater network works</td>
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<td></td>
<td>Telecommunication works</td>
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<td>Irrigation works</td>
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<td>Landscaping and street</td>
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</tr>
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<td>CCTV cameras, irrigation</td>
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</table>

*A full list of activities involved in the construction and maintenance phases are generally listed in the BOQ of road projects*

2.2 DATA AND STUDY AREA

Based on the type and posted speed limit, three roads were selected in the Abu Dhabi Emirate for the case study to estimate their carbon footprint using the RoadCO₂ model. The locations of these roads are shown in Figure 2. The three cases are:

- Case study 1: Construction of internal roads and services network in Al Rahba City.
- Case study 2: Upgrading of Al Salam Street to an expressway and construction of the tunnel.
- Case study 3: Widening of Eastern Corniche Road.

Case Study 1: Al Rahba city internal roads and services network construction project includes the construction of internal roads and services. It is the urban local road with a posted speed limit of 60 km/hr. The project is located in the city of Al Rahba along the Abu Dhabi – Dubai highway. It consists of the construction of a 30-km single carriageway, 7.30 m wide, two-way traffic flanked by 0.35 m wide outer shoulders and by 2.00 m wide footpaths. The construction also includes parking areas, footpaths, protection and relocation of existing services, ducts for future utility crossing, street lighting works, storm water drainage works and ancillary works.

Case Study 2: Upgrading of Al Salam Street to an expressway involved the construction of a tunnel from the Dalma Street to the Corniche and Mina Port. In addition to the tunnel construction, surface roads were widened and several interchanges were constructed. The Sheikh Zayed tunnel is 3.6 km long with 4 lanes on each side. The posted speed limit of the road ranges from 60 to 80 km/hr. The project involves four sections running from Al Salam Bridge to Al Mina Street and consists of five tunnels, a bridge, bypasses, surface and service roads.

Case Study 3: The scope of this project covers widening of the existing Eastern Corniche Road from IP69 (Station 15+750) to IP98 (Station 15+200) by adding a fourth traffic lane of 3.65 m and a shoulder of 3 m in each direction along with all the necessary works. Since it involved the widening of an existing road, a temporary detour of the traffic and free flow of traffic was considered on the road. The basic construction included an extra lane and a shoulder with a length of 2.87 km while moving the sub-surface utilities to the new location.

The data for the three case studies were obtained from the Abu Dhabi City Municipality (ADM). The quantity of construction materials was extracted from the BOQ provided by ADM. The activities mentioned in the BOQ of the three case study roads were thoroughly analyzed and arranged to be entered in the RoadCO₂ model (Figure 1). In particular, data were arranged into three categories (materials, transportation and equipment) to be utilized as resources in the developed model (Figure 1 and Table 1). For the transportation of materials to the construction site, local mode of
transport was considered. Fuel consumption and travel distance were estimated based on consultation with expert consultants and contractors. Data for the operations of equipment (rollers, pavers, etc.) were extracted from the BOQ.

3 RESULTS AND DISCUSSION

3.1 EMISSIONS FROM THE CONSTRUCTION PHASE

Construction projects have a significant impact on the environment. As natural resources are being exhausted, fossil fuels are emitting damaging pollutants, and rainforests are being destroyed, considerations of how much construction impacts the environment is becoming a topical issue (Finch, 1992). The initial impact of a facility on the environment results from the energy and products consumed in its construction, and thereafter the facility continues to affect the environment throughout its operation, maintenance, refurbishment, and, finally demolition (Treloar et al., 2004). The GHG emissions for the three case study roads, presented earlier, were estimated using the developed RoadCO\textsubscript{2} model. The outcomes of the three case studies are described in this section.

Case Study 1: The construction of this project also includes construction of a sub-surface storm water network. The estimated total emission from materials used was 9766 ton CO\textsubscript{2} eq. Figure 3 shows the relative contribution of each category in the total emission. Materials used in the storm water drainage system contributed the highest (39\%) to GHG emissions among the eight categories. This is reasonable given that it was the establishment of new infrastructure for a newly planned residential city. GHG emissions from the storm water drainage system is followed by the road works (15\%), irrigation (12\%), and sewerage network (11\%).

Diesel was used in all equipment to carry out the site activities. The estimated total emission from equipment used in the construction phase was about 22712 ton CO\textsubscript{2} eq. Figure 4 shows the relative contribution of different equipment used in the construction. Among the equipment used in the case study 1, the highest emission was estimated for compactors (4627 ton CO\textsubscript{2} eq.). Compactors were used in the compaction of sub-base and base grade as well as asphalt pavement. The total road network length is 30 km, which justify the highest emissions from the compactors. Loaders came in the second place and contributed about 4606 ton CO\textsubscript{2} eq. The 3\textsuperscript{rd} highest emission was estimated from the excavators, followed by the tippers in the 4\textsuperscript{th} place.
**Figure 3.** Emissions from materials used in Al Rahba project (case study 1).

**Case Study 2:** Upgrading of Al Salam Street includes the construction of a tunnel of length 3.6 km with 4 lanes on each side. The tunnel is a concrete structure with three interchanges. The estimated total emission from the materials used in the construction phase was about 282745 ton CO$_2$eq. Figure 5 shows the relative contribution of each material type used to the total emission. The highest emission is from concrete which is about 77% of the total. The 2nd and 3rd highest emissions are estimated from the steel 14% and asphalt & GRP 4%.

**Figure 4.** Emissions from equipment used in Al Rahba project (case study 1).
Figure 6 shows the emissions from equipment used in the construction phase. Total emission from the equipment was about 50427 ton CO$_2$ eq. The highest emission (21312 ton CO$_2$ eq) is found from the trucks used to transport concrete and other materials to the site. The 2$^{nd}$ highest emissions (3412 ton CO$_2$ eq) are estimated from the soil compactor, and the 3$^{rd}$ highest are the excavators (3204 ton CO$_2$ eq).

**Case Study 3**: The total emission from materials used in the widening of the Eastern Corniche Road was about 9,533 ton CO$_2$ eq. Figure 7 shows the relative contribution of each category to the total emissions. The highest emission is estimated from road works, which is about 55% of the total emissions. Both lighting & electrical works and
Irrigation network works comes in second and both contribute 9%. An already existing sub-surface network of utilities was moved to a new location, thereby an increased emission is estimated from the concrete works. Both water network works and stormwater network works contribute about 8%, followed by the telecommunication works (7%).

![Emissions from materials used in the widening of the Corniche Road (case study 3).](image)

The estimated total emission from equipment used in the construction was about 2466 ton CO\textsubscript{2} eq. Figure 8 shows the relative contribution of different equipment used in the construction phase. Front shovels are the top emitter with emissions of 423 ton of CO\textsubscript{2} eq, followed by the excavators, which emitted about 289 ton CO\textsubscript{2} eq. Substantial excavation work was carried out by the equipment to move the sub-surface utilities to a new location.

![Emissions from equipment used in widening the Corniche Road (case study 3).](image)
Interpretation and Comparison of Results

The results from the three case studies are shown in Table 2, along with a similar case study conducted in the UAE using the CHANGER software. Since the roads in Al Rahba City (case study 1) are internal conventional roads, so the normalized emissions (i.e., 541 ton CO\(_2\) eq/km/lane) from their construction are relatively low. Their pavement design, load bearing capacity and posted speed limit are less as compared to other case studies. The Al Rahba (case study 1) construction involves more activities on the stormwater drainage works. The upgrading of Al Salam Street (case study 2) includes construction of a tunnel and a road. Its normalized emission is the highest among all the three projects due to a tunnel construction (total normalized emission 11568 ton CO\(_2\) eq/km/lane). The Corniche project (case study 3) is similar to a previous case study conducted by Huang et al. (2013), which involved addition of lanes on an existing road. The normalized emissions from the Corniche project (case study 3) (1045 ton CO\(_2\) eq/km/lane) and from the Huang et al. (2013) road case study (2140 ton CO\(_2\) eq/km/lane) are comparable. It should be stressed that the final results cannot be properly understood without considering the scope established by researchers.

Table 2. Comparison of construction phase emissions between the three case studies

<table>
<thead>
<tr>
<th>Case Study</th>
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<th>Number of Lanes</th>
<th>Materials (tCO(_2)eq/km/Lane)</th>
<th>Construction Equipment (tCO(_2)eq/km/Lane)</th>
<th>Total (tCO(_2)eq/km/Lane)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (Al Rahba)</td>
<td>30.00</td>
<td>2</td>
<td>163</td>
<td>379</td>
<td>541</td>
</tr>
<tr>
<td>2 (Al Salam)</td>
<td>3.60</td>
<td>8</td>
<td>9818</td>
<td>1751</td>
<td>11568</td>
</tr>
<tr>
<td>3 (Corniche)</td>
<td>2.87</td>
<td>2</td>
<td>830</td>
<td>215</td>
<td>1045</td>
</tr>
</tbody>
</table>

3.2 EMISSIONS FROM THE OPERATION PHASE

Estimation of traffic emissions has become increasingly relevant in the discussion of air quality problems, climate change and mitigation policies, due to the continued growth in vehicle use and the deterioration in driving conditions (Smit et al., 2010). The road operation stage has been found to be the main producer of GHG emissions during a road project’s life-cycle (ADB, 2010). Given the relevance of road operation to GHG emissions, one needs to craft a methodology capable of quantifying these emissions. In order for one to approach this problem, a number of variables need to be considered such as travel road length, traffic volume, GHG content of different fuel types, as well as the number of kilometers traveled by the entire vehicle and fuel efficiency rate for each vehicle class. The developed RoadCO\(_2\) model has the capability to quantify the operational phase emissions based on these variables. Because of the availability of traffic data, the model was used to quantify the operational phase emissions for two case studies only (case studies 1 and 2). Table 3 shows the variables that were used as inputs in the RoadCO\(_2\) model.

Table 3. Variables used for the operation phase emission estimation (ADM, 2016)

<table>
<thead>
<tr>
<th>Traffic Volume</th>
<th>Travel Road Length</th>
<th>Al Rahba Internal Roads (Case study 1)</th>
<th>Al Salam Street (Case study 2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Annual traffic</td>
<td>AADT</td>
<td>Annual traffic</td>
</tr>
<tr>
<td>1.724 km</td>
<td>Passenger Cars</td>
<td>3,784,320</td>
<td>10,368</td>
</tr>
<tr>
<td></td>
<td>Light Vehicle</td>
<td>206,955</td>
<td>567</td>
</tr>
<tr>
<td></td>
<td>Heavy Vehicle</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2.18 km</td>
<td>Passenger Cars</td>
<td>6,524,168</td>
<td>17,874</td>
</tr>
<tr>
<td></td>
<td>Light Vehicle</td>
<td>356,790</td>
<td>978</td>
</tr>
<tr>
<td></td>
<td>Heavy Vehicle</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

AADT = Annual Average Daily Traffic

Another factor affecting the amount of emissions produced by road operation is the type of fuel used by the operating vehicle fleet. Usually vehicle fleet comprises of different vehicle types consuming different fuel types. Depending on their composition, different fuel types produce different amounts of emissions in terms of carbon-equivalent. Another major factor needed to quantify the amount of GHG emitted is fuel efficiency. Fuel efficiency rates for each vehicle class are also needed in order to determine the amount of fuel consumed (in liters) while vehicles travel on a road. The RoadCO\(_2\) model provides a range of fuels and fuel efficiencies that can be used in the calculations. Though some of these information and rates used in the RoadCO\(_2\) model may be determined from the literature, they still can produce an estimation to the road operation emissions that is close to the real life situation. Tables 4 and 5 show the GHG content of the three fuels commonly used in the Emirate of Abu Dhabi, and fuel efficiencies used to estimate the GHG emission from the two case studies, respectively.
Table 4. GHG content of the three types of fuels commonly used in Abu Dhabi Emirate

<table>
<thead>
<tr>
<th>Fuel Type</th>
<th>CO₂ (kg/Liter)</th>
<th>CH₄ (kg/Liter)</th>
<th>N₂O (kg/Liter)</th>
<th>Carbon Equivalent (kg/Liter)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gasoline</td>
<td>2.38</td>
<td>0.000034</td>
<td>0.000020</td>
<td>2.39</td>
</tr>
<tr>
<td>Diesel</td>
<td>2.66</td>
<td>0.00107</td>
<td>0.000021</td>
<td>2.67</td>
</tr>
<tr>
<td>Liquified Natural Gas</td>
<td>1.43</td>
<td>0.000067</td>
<td>0.0001332</td>
<td>1.44</td>
</tr>
</tbody>
</table>

Table 5. Fuel efficiency rates considered in the RoadCO₂ model

<table>
<thead>
<tr>
<th>Vehicle Class</th>
<th>Fuel Efficiency Rate (L/km) – City</th>
<th>Fuel Efficiency Rate (L/km) – Highway</th>
<th>Fuel Efficiency Rate (L/km) – Combination*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passenger Car</td>
<td>0.1196</td>
<td>0.084</td>
<td>0.104</td>
</tr>
<tr>
<td>Light Vehicle</td>
<td>0.1709</td>
<td>0.125</td>
<td>0.150</td>
</tr>
<tr>
<td>Heavy Vehicle</td>
<td>-</td>
<td>0.375</td>
<td>0.375</td>
</tr>
</tbody>
</table>

*55% City, 45% Highway

Table 6 shows the total annual amount of emissions, in terms of ton CO₂ eq, produced by the road segment in Al Salam Street (case study 2). The total annual amount reaches over 37000 tons. The total fuel consumed shown in the third column, from right to left, takes into consideration both the kilometers traveled by the entire fleet (Table 3), and the fuel efficiency rates shown in Table 5. A combination pattern was used for Al Salam Street. Finally, the carbon-equivalent emissions in tons, shown in the rightmost column, produced by each vehicle class is determined by multiplying the total amount of fuel consumed by each vehicle category by the emission factors contained in Table 4. Similarly, Table 7 shows the total annual amount of emissions, in terms of ton CO₂ eq, produced by the road network within the boundaries of Al Rahba city (case study 1). In the calculation, it was assumed that all passenger cars use gasoline, whereas all heavy vehicles use diesel. As for the light vehicles, the assumption was made that 60% use gasoline and the rest 40% use diesel.

Table 6. Carbon emission due to vehicle operation on Al Salam Street segment (case study 2)

<table>
<thead>
<tr>
<th>Vehicle Class</th>
<th>Travel Road Length (km)</th>
<th>Total Fuel Consumed (L)</th>
<th>Emission (ton CO₂ eq)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passenger Car</td>
<td>2.18</td>
<td>262163</td>
<td>6266</td>
</tr>
<tr>
<td>Light Vehicle</td>
<td>-</td>
<td>603476</td>
<td>16113</td>
</tr>
<tr>
<td>Heavy Vehicle</td>
<td>-</td>
<td>463390</td>
<td>15385</td>
</tr>
</tbody>
</table>

Table 7. Carbon emission due to vehicle operation on Al Rahba city road network segment (case study 1)

<table>
<thead>
<tr>
<th>Vehicle Class</th>
<th>Travel Road Length (km)</th>
<th>Total Fuel Consumed (L)</th>
<th>Emission (ton CO₂ eq)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passenger Car</td>
<td>1.72</td>
<td>780290</td>
<td>1865</td>
</tr>
<tr>
<td>Light Vehicle</td>
<td>-</td>
<td>60975</td>
<td>163</td>
</tr>
<tr>
<td>Heavy Vehicle</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

It is estimated that the traffic operating on the 2.18-km section on Al Salam Street produces over 37000 ton CO₂ eq annually, whereas traffic operating on the road network within Al Rahba city produces approximately 2000 ton CO₂ eq. This significant difference is in line with the major dissimilarities between these two transportation facilities. Al Salam Street is a high traffic volume, multilane road carrying large number of heavy vehicles such as trucks and buses. On the other hand, Al Rahba city network is mostly accessed by residents (passenger cars), which translates into much lighter traffic volume and no heavy-vehicle traffic is allowed within the city boundaries. The comparison of their emissions is shown in Figure 9. Table 3 indicates that heavy-vehicle traffic accounts for only 16% of the total traffic on the Al Salam Street and Table 6 indicates that it produces about 40% of the total emissions. On the other hand, passenger cars account for over 30% of the total traffic on Al Salam street, but they produce less than 20% of the total emissions. It is important to clarify here that emission from the operation phase were estimated considering the annual traffic volume only. Other issues that result in emission during the operation phase have not been considered, including emission caused by utilization of electricity for lighting, traffic signals and for irrigation. Also, to be able to estimate the
total emissions associated with the operation phase, one needs to consider the life span of roads as well as variations in the traffic volume over time.

Figure 9. Comparison of annual GHG emissions due to vehicle operation on Al Salam Street segments and Al Rahba city internal road segment

4 CONCLUSION

The transportation sector is one of the fastest growing main contributors to global climate change. It accounts for more than 23% of energy-related CO\textsubscript{2} emissions and other GHG emissions. Road life cycle includes several phases, namely pre-feasibility, feasibility, engineering design, construction, operation and maintenance. GHGs are emitted in all road phases whether in a direct or an indirect way. The carbon footprint of a road project is generally conducted in its construction, operation and maintenance phases. It is important to understand all the possible sources of direct and indirect CO\textsubscript{2} emissions during the full life cycle of a road.

The RoadCO\textsubscript{2} model was developed and used to estimate GHGs emissions from three road construction projects located in the Abu Dhabi Emirate. These case studies are (1) Al Rahba city internal roads and services (2) upgrading of Al Salam Street to an expressway, and (3) widening of the Eastern Corniche road. The normalized emissions from the construction of these case studies were found to be 541, 11568, and 1045 ton CO\textsubscript{2} eq/lane/km, respectively.

In the Al Rahba city internal roads and services project, materials used in the storm water drainage system contributed the highest emissions (39%), followed by road works (15%) and irrigation (12%) and sewerage network (11%). Among the equipments used in Al Rahba case study, the highest emission was estimated from the compactors (4627 ton CO\textsubscript{2} eq.), followed by loaders (4606 ton CO\textsubscript{2} eq), excavators (2770 ton CO\textsubscript{2} eq) and tippers (2400 ton CO\textsubscript{2} eq). The upgrading of Al Salam Street to an expressway project includes the construction of a tunnel of 3.6 km. The highest material emission was found from the concrete (77% of total emissions), followed by steel (14%) and asphalt (4%). In the case of equipment, the highest emission (21312 ton CO\textsubscript{2} eq) was found from the trucks used to transport concrete and other materials, followed by soil compactors (3412 ton CO\textsubscript{2} eq). For the Eastern Corniche road widening project, the highest emission was estimated from road works (55% of total emissions), followed both lighting & electrical works and Irrigation network works (9%). In the case of emissions from equipments, the highest emission was found from the front shovels (423 ton CO\textsubscript{2} eq), followed by excavators (289 ton CO\textsubscript{2} eq).

In the operation phase, the traffic operating in the 2.18-km section of Al Salam Street project produces over 37763 ton CO\textsubscript{2} eq/year, whereas traffic operating on the road network within Al Rahba city produces approximately 2028 ton CO\textsubscript{2} eq/year. This significant difference is in line with the major dissimilarities between the two transportation facilities. Emissions from passenger cars dominate in Al Rahba city internal roads, whereas light and heavy vehicles dominate in the case of Al Salam Street.

The UAE specific emission factors are not available yet, therefore the developed model uses IPCC Tier 1 default emission factors. It is highly recommended to use local emission factors, when available, for more accurate emissions estimation. It is also a key recommendation of the Abu Dhabi GHG emissions inventory (EAD, 2012) to develop local emission factors in the UAE. The RoadCO\textsubscript{2} model is designed in such a way that it can be updated with local emission factors once available.
5 ACKNOWLEDGEMENTS

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REFERENCES


ADM (2016) Traffic counts data provided by the Abu Dhabi City Municipality, personal communication.


**PAPER TITLE**
MICRONIZED RECYCLED CONCRETE AS AN ALTERNATIVE BINDER/ACTIVE FILLER FOR STABILIZED AND COLD RECYCLED MIXTURES

**TRACK**
5.2: Greening Road Projects

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<th>POSITION</th>
<th>ORGANIZATION</th>
<th>COUNTRY</th>
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<tbody>
<tr>
<td>Zuzana CIZKOVA</td>
<td>Researcher at Department of Road Structures</td>
<td>Faculty of Civil Engineering, Czech Technical University in Prague</td>
<td>Czech Republic</td>
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<tr>
<td>Jakub SEDINA</td>
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<tr>
<td>George KARRA´A</td>
<td>CEO</td>
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</tr>
</tbody>
</table>

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**KEYWORDS:**
Mechanically activated material, recycled concrete, high-speed milling, cold recycled mix, stabilized granular material mixtures

**ABSTRACT:**
This paper summarized experimental results of a three-year research and innovation development, which focused on the possible reuse of recycled concrete by mechanically activated small-sized particles of such material. The fine-grained recycled concrete (FGRC) was used as a partial or full substitute of cement in stabilized pavement mixtures which are traditionally bond or co-bond by such hydraulic binder. This application is therefore suitable either for cement stabilized granular materials or cold recycled asphalt mixtures. Characterization of such mixture variants was done by selected strength and deformation characteristics including either water susceptibility assessment or resistance to frost cycles. Material analyses of micronized recycled concrete is discussed as well as the performance of the final mixture including its comparison to traditional stabilized granular materials or cold recycled asphalt mixtures.
Micronized recycled concrete as an alternative binder/active filler for stabilized and cold recycled mixtures

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1 INTRODUCTION

Cement is an important binder for both cold recycled and stabilized mixtures used in pavement structures. Adding cement improves moisture susceptibility and provides sufficient strength and stiffness especially during the early stage of mix curing. On the other hand, adding cement to cold recycled or stabilized mixtures is quite costly and is always associated with environmental concerns related to its production. The negative aspects could be mitigated if it is replaced – whether partly or in full – by mechanical-chemically activated material on the basis of recycled concrete, or its fine fractions.

The positive effect of the idea to substitute cement by pulverized recycled material on the environmental and economical matter is obvious. That is the reason why the use of micro-milled materials has been investigated in the laboratory of road structures at CTU in Prague for more than 3 years. Some results have been already presented, for example in [Suda 2016, Cizkova 2016]. The main objective of this research was to find the most convenient mix composition which provides economic and environmental benefits as well as sufficient strength and other important material qualities, like e.g. moisture or frost susceptibility. This paper summarizes all the important findings of this research. The research was focused mainly on mechanically activated recycled concrete, but in the beginning micro-milled limestone was investigated as well. Results of pulverized limestone testing are included in this paper as well, but further investigation of this topic wasn’t performed, because this material didn’t show any beneficial effects if compared e.g. to lime hydrated. The investigation was focused mainly on the cold recycled mixtures and so represents the majority of this paper, but there is also a chapter summarizing the results of pulverized materials addition into the stabilized materials at the end of this contribution.

1 HIGH SPEED MILLING

The technological aspects of industrial production determine some advanced solutions and a need for material disintegration. Such aspects are currently present in a broad range of industries, from food to chemical and metallurgical industries. Material activation in high-speed or high-energy mills is a process where the solid phase particles are broken mechanically and the specific surface increased, thus causing a significant change in its granularity. The changes affect chemical and physical properties of the ground material.

A mechanical process or intervention with the structure of a material that increases its chemical reactivity can be called mechanical activation. Such an intervention in the structure of the substance can be achieved through grinding. According to the traditional interpretation, grinding is defined as mechanical dispersion of solids which results in reduced particle size at the same time with increased specific area and surface energy within the framework of the entire material system; nevertheless, the mechanical effects occurring throughout dry grinding of solids might also launch considerable structural changes and chemical reactions in the materials ground. The character of the surface, i.e. its morphology, distribution of charges, chemical nature of the thin surface film of the particle etc., also has a significant effect [Faltus 2009; Sekulič 1999; Blanco 2006].

This paper concerns an experimental assessment of mechanically activated pulverized concrete from reclaimed cement concrete pavements and mechanically activated pulverized limestone. Such modified material was applied as a partial substituent for standard hydraulic binder or as active microfiller in cold recycling mixes; its impact on the resulting strength and deformation parameters of the mix was observed. Last but not least, moisture and frost susceptibility of such mixtures were tested as they are determining the overall pavement lifetime. The experience with the use of reclaimed materials as a binder or its components has not been as extensive so far as to allow drawing general conclusions on practical application options.
2 MATERIALS

The research was focused mainly on cold recycled mixes, therefore almost all designed mixes contained the same type of screened reclaimed asphalt material (RAP) with 0/22 mm grading originating from the same source (mix asphalt plant Středokluky - see Figure 1). Nevertheless, the homogeneity of RAP was quite poor, which is typical not only for the Czech circumstances. More generally it is typical for these materials if selective cold pavement milling for each construction site is not done. This fact influenced greatly the test results and complicates always setting of any final conclusions with appropriate repeatability of determined data. The bitumen content in RAP was determined to be 5.6 % by mass. Nevertheless, this value should be considered as just approximate because the composition of RAP differed even within a single batch not to mention the difference between batches. Because of that it was very important to perform all measurements for each set at once. Measuring of some related values later using specimens made from another batch is not recommendable because the RAP composition influences greatly the final mix characteristics, as is discussed by [Valentin 2014]. Because of its heterogeneity, the RAP was assessed repeatedly in terms of its grading and the results are shown in Figure 1 including the grading limits according to the Czech technical specifications [TP 208].

Figure 1. Grading curves of used RAP 0/22 mm (location Středokluky, repeated analyses)

The used cement was classified as CEM II / B 32.5 R according to [EN 197-1]. The optimal moisture content of the cold recycled mix was determined according to [EN 13286-2] and reflected already the presence of cement in the final cold recycled mix.

The used bituminous emulsion can be characterized as a cationic slow-breaking bituminous emulsion C60B8 according to the designation in [EN 13808] which is commonly used in the Czech Republic (Europe). For the production of foamed bitumen standard paving grade bitumen 70/100 was applied according to [EN 12591]. When preparing the foamed bitumen, there was 3.8 % of water added to the bitumen (according to the procedure recommended for cold recycling technology by [Wirtgen Manual, 2012]). Foamed bitumen was injected into the cold recycled mix under the temperature between 160 °C and 170 °C using the Wirtgen WLB10S laboratory equipment.

Table 1. Basic parameters of finely ground material

<table>
<thead>
<tr>
<th>Sample after mechanical activation</th>
<th>Reclaimed concrete</th>
<th>Crystalline limestone</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (g/cm³)</td>
<td>2.670</td>
<td>2.818</td>
</tr>
<tr>
<td>Specific surface (m²/kg) according to Blaine</td>
<td>435</td>
<td>455</td>
</tr>
</tbody>
</table>

Figure 2. Granular analysis of crystalline limestone and reclaimed concrete by laser granulometry
The conducted experiments focused on the application of two kinds of mechanically activated materials, namely finely ground/pulverized reclaimed concrete (FGRC) and finely ground/micro-pulverized limestone (FGML). Large pieces of concrete reclaimed from a cement concrete pavement of backbone Czech motorway D1 during its reconstruction were crushed to fraction 0/22 mm in the laboratories at CTU Prague. Pulverized crystalline limestone was obtained as waste from marble processing in Palestine. The materials were crushed in an impact crusher to fraction < 1 mm. Intense mechanical activation, or intense grinding in a high-speed mill with repulsion disc rotors followed. The mutual perimeter speed of counter-rotating rotors was set to approx. 200-250 m/s. Based on the assumptions, the mode was supposed to guarantee a very high rate of material refinement with acceptable economy (high productivity, reasonable energy consumption, low wear of machinery). Within the proposed mode, only a significant increase in specific surface should have been observed (see Table 1).

The comparison of results of laser granulometry suggests the conclusion that high-speed grinding achieved a very fine grinding of the input material to prevailing particle size of 0.5 – 50 μm, where the average grain was 10-15 μm (see Figure 2).

Cylindrical specimens with 150±1 mm diameter and 60 mm were compacted by static pressure of 5.0 MPa were prepared. To simulate the initial moisture content of the mixture after paving and compaction of the fresh mix, specimens were stored for the first 24 hours at 90-100 % relative humidity and a temperature of (20±2) °C. This was done by keeping the specimens in the mould or by putting them into a suitable plastic bag. Further the specimens were stored at laboratory conditions with 40-70 % relative humidity and a temperature of (20±2) °C for the rest of their curing time. The volumetric parameters and indirect tensile strength (ITS) of the test specimens were determined according to (TP 208). For all test specimens stiffness modulus data were collected. Repeated indirect tension to cylindrical specimens test (IT-CY) was used. Measurement of indirect tensile strength and stiffness modulus was performed at 15 °C after 7, 14 and 28 days of specimen curing.

For all of the cold recycled mixes moisture susceptibility was determined with derived indirect tensile strength ratio (ITSR) according to [TP 208]. The test specimens that cured for 7 days at laboratory conditions were suspended in water of (20±2) °C for another 7 days and then used for the ITS test. Such prepared specimens were subjected to non-destructive stiffness testing by the IT-CY method prior to ITS testing to determine the effects of water on mix stiffness.

The specimens from stabilized mixes were prepared in the Proctor compactor according to [ČSN EN 13286-50]. Due to the material grain size, a Proctor Standard mould with a diameter of 150 ± 1 mm and a height of 120 ± 1 mm was chosen. The modified proctor compaction was performed, which corresponds to the compaction of the base layers. Optimal moisture content was determined according to [EN 13286-2]. After compaction, the specimens were preserved according to the requirements of [ČSN EN 14227-1] for 28 days in sealed conditions at the temperature of 20 ± 2 °C. After that, specimens were tested for compressive strength according to [ČSN EN 13286-41] and resistance to frost and water. The testing itself was carried out using a static compactor where the specimen is continuously loaded and its failure occurs from 30 to 60 seconds after the beginning of loading. The compressive strength R,c is determined as a ratio of the maximum force recorded at the specimen failure and the cross sectional area of the tested specimen. The classification is then carried out according to [ČSN EN 14227-1].

For the frost-heave test according to [ČSN EN 14227-1], the specimens were subjected to 13 freezing cycles (18 hours at the laboratory temperature of 20 - 25 °C and 6 hours in the climate chamber at -20 ± 2 °C). These conditions correspond to the requirements for the base layer. The compression strength according to [ČSN EN 13286-41] of such treated specimens should achieve at least 85% of R,c measured after 28 days of specimen curing at the laboratory conditions.

### 3 MIX DESIGN

| Table 2: Composition of the tested cold recycled mixes |
|---------------------------------|----------|----------|----------|----------|----------|----------|----------|----------|
| RAP 0/22 | 3.5E+3C | 4.5F+3C | 3.5E | 4.5F | 3.5E+5FGRC | 3.5E+3FGRC | 3.5E+5FGML | 3.5E+3FGML |
| Water     | 91.5    | 90.5    | 94.5    | 93.4    | 89.0    | 91.5    | 89.0    | 91.5    |
| Bituminous emulsion | 2.0    | 2.0    | 2.0    | 2.0    | 2.5    | 2.0    | 2.5    | 2.0    |
| Foamed bitumen | 3.5    | 3.5    | 3.5    | 3.5    | 3.5    | 3.5    | 3.5    | 3.5    |
| Cement   | 4.5    | 4.5    | 4.5    | 4.5    | -      | -      | -      | -      |
| FGRC   | 3.0    | 3.0    | 3.0    | 3.0    | -      | -      | -      | -      |
| FGML | 5.0    | 3.0    | 5.0    | 3.0    | -      | -      | -      | -      |
28 different mix designs of cold recycled mixtures were selected and tested (table 2). Mix 1F+1C+4FGRC was prepared twice (1F+1C+4FGRC and 1F+1C+4FGRC) because the bituminous binder used for preparation of foam bitumen in case of the mix 1F+1C+4FGRC showed already some binder ageing degradation caused by its repeated heating.

Table 3. Mix design of stabilized mixes

<table>
<thead>
<tr>
<th>Mix</th>
<th>Crushed stone 0/32 (Zbraslavl)</th>
<th>Gravel 0/16 (Uhy)</th>
<th>Optimal moisture content</th>
<th>Amount of Cement</th>
<th>Amount of micromilled concrete</th>
<th>Amount of micromilled concrete + limestone</th>
</tr>
</thead>
<tbody>
<tr>
<td>V1</td>
<td>80.0 %</td>
<td>20.0 %</td>
<td>6.2 %</td>
<td>6.0 %</td>
<td>0.0 %</td>
<td>0.0 %</td>
</tr>
<tr>
<td>V2</td>
<td>80.0 %</td>
<td>20.0 %</td>
<td>6.2 %</td>
<td>3.0 %</td>
<td>6.0 %</td>
<td>0.0 %</td>
</tr>
<tr>
<td>V3</td>
<td>80.0 %</td>
<td>20.0 %</td>
<td>6.2 %</td>
<td>1.5 %</td>
<td>9.0 %</td>
<td>0.0 %</td>
</tr>
<tr>
<td>V4</td>
<td>80.0 %</td>
<td>20.0 %</td>
<td>6.2 %</td>
<td>4.0 %</td>
<td>2.0 %</td>
<td>0.0 %</td>
</tr>
<tr>
<td>V5</td>
<td>80.0 %</td>
<td>20.0 %</td>
<td>6.2 %</td>
<td>0.0 %</td>
<td>0.0 %</td>
<td>6.0 %</td>
</tr>
</tbody>
</table>

The composition of the tested stabilized mixes is listed in table 3. It was based on the requirements of European standards for granular composition and strength characteristics of mixes SC C₅₀. These mixtures achieve strength classes of at least C₅₀, while at the same time showing sufficient resistance to frost and water effects. Higher contents of alternative binders were proposed with respect to previous experience and potential cost, which should be roughly half of the cost of the commonly used mixed cements.
DISCUSSION OF RESULTS

The overall results of indirect tensile strength, stiffness modulus and moisture susceptibility are summarized in figures 3-5. The mixes are divided into 3 groups distinguished by colour. That is because the mixes from each group were prepared form the same batch of RAP. As a result of the RAP heterogeneity discussed earlier, the comparison of mixes from different batches is questionable. The values of moisture susceptibility are shown compared to the thresholds defined in [TP 208] – 70 % for mixes with bituminous and hydraulic binder and 60 % for mixes including just bituminous binder. The amount of measured data is too big not to get lost, therefore some important findings are demonstrated on some chosen mixes in the following subchapters.

Generally, it can be stated that adding of recycled concrete into the cold recycled mixes causes slightly higher stiffness and comparable or slightly lower tensile strength. Nevertheless, the biggest influence of adding recycled concrete is on moisture susceptibility. The addition of micro-milled limestone into the cold recycled mixtures didn’t show any significant advantages, therefore it was not further investigated. It should be stated that there was not some excessive expectations and the results just confirm, that pulverized (micro-milled) limestone from waste dust or sludge can be used as an alternative e.g. to traditional lime hydrated.

The ITS and stiffness values of the mix with new binder (1F+1C+4FGRC) are in average about 20% higher than those of the mix in which the asphalt binder used for the foamed bitumen production was partially degraded by the aging process (1F+1C+4FGRC’).

Figure 3. Indirect tensile strength of all tested cold recycled mixes

Figure 4. Stiffness modulus of all tested cold recycled mixes
4.1 THE IMPACT OF ADDED PULVERIZED CONCRETE ON MOISTURE SUSCEPTIBILITY

Figure 6 depicts the indirect tensile strength and moisture susceptibility of several tested mixes. The figure shows that all tested mixtures comply with the required minimum ITS value 0.3 MPa according to [TP 208] but the mixture 2F+3C with 3% of cement exceeds the maximum value 0.7 MPa. In order to avoid the risk of hydration cracks origin, this threshold should not be neglected. Lower ITS values of mixtures with micro-milled concrete are therefore better than the results of more expensive reference mix with 3% of cement, which does not meet the requirements defined in [TP 208]. In other words, there is a way to process a large amount of waste concrete which, when applied in cold-recycled mixtures, causes slightly higher stiffness and comparable or slightly lower tensile strength. The ITS decrease is not significant and the mixes with pulverized concrete meet the requirements of [TP 208].

Nevertheless, the unquestionable, repeatedly proven reason for development of the high-speed milling technology of recycled concrete is its impact on the BSM’s moisture susceptibility. As can be seen from figure 6, all mixtures containing cement or micro-milled concrete fulfil the moisture susceptibility criteria of [TP 208] – 70% for mixes with bituminous and hydraulic binder and 60% for mixes including just bituminous binder. Mixes with micro-
milled concrete achieve even higher values of moisture susceptibility than mixtures with cement. On the other hand, very low values of moisture susceptibility were measured for the mix 2F, which does not contain any hydraulic binder, although the limit is still tightly fulfilled.

The comparison of cold recycled mixture 2P+3MB with 3 % of pulverized concrete and the mix 2P+1C with 1 % of cement, is also very interesting. Both mixtures achieve very similar results. The ITS values of both mixtures measured on specimens cured for 7 days at laboratory conditions and for additional 7 days in water are nearly the same (0.42 MPa in 2P+3MB and 0.41 MPa at 2P+1C). In terms of expressing the ITSR value, the mix 2P+3MB achieves even higher value due to the lower ITS value after 7 days. Additionally, during the first 28 days of specimen curing, the ITS value of the 2P+3MB mixture grew more than ITS value of the 2P+1C mixture (2P+3MB reached 0.68 MPa, 2P+1C reached 0.62 MPa). To conclude, the only characteristic of cold recycled mixtures which is better in mixtures with cement than in mixtures with the pulverized concrete is the stiffness. However, [TP 208] does not have any requirement on stiffness of cold recycled mixtures. In terms of ITS and moisture susceptibility, these mixtures with micro-milled concrete are not only better than mixtures without any hydraulic binder but in many cases even slightly better than mixtures with cement.

4.2 THE INFLUENCE OF FOAMED BITUMEN CONTENT

When comparing similar mixtures, which differ just in the foamed bitumen content, it can be seen that mixtures with 2 % of foamed bitumen reach higher results of both measured characteristics than mixtures with 1 %. Similarly, higher results were achieved by mixtures with 4 % of foamed bitumen than by mixtures with 2 %. This phenomenon is clearly visible from figures 7 and 8 that depict 3 pairs of mixtures differing just in the foamed bitumen content. The mix with higher content of foamed bitumen (depicted with lighter colour) achieves higher strength and stiffness in all 3 cases. Also the mixes depicted in figures 3 and 4 by the green colour achieve generally lower (but still sufficient) values of ITS and stiffness modulus than mixes tested earlier depicted by the blue colour, because those mixes contained higher amount of bituminous binder.

![Figure 7: Indirect tensile strength of three pairs of mixes with different foamed bitumen content](image)

![Figure 8: Stiffness modulus of three pairs of mixes with different foamed bitumen content](image)
In the past it was discovered, that adding of foamed bitumen in higher amount than the optimum leads to a mix which is too flexible. Mixtures with more than 2.5 % of foamed bitumen had usually lower strength and stiffness than mixtures with lower amount of foamed bitumen [Valentin 2016]. The presented finding of better strength and stiffness connected with increasing foamed bitumen content is therefore in contrast to [Valentin 2016]. The explanation for that can be that it is caused by the RAP heterogeneity discussed earlier. Another possible reason for that can be the modification of the grading curve of a cold-recycled mix as a result of adding the finely ground concrete into the mix. The higher content of fine particles can cause the need of higher amount of bituminous binder in order to form „mortar” together, which then coats bigger grains. The optimal foamed bitumen content can therefore move higher.

4.3 THE INFLUENCE OF PULVERIZED CONCRETE CONTENT

Figures 9 and 10 are focused on the influence of added pulverized concrete content. Unfortunately, the influence of added pulverized concrete isn’t unambiguous, but the best qualities are achieved usually by the mix 2F+1C+4PC. Despite the positive effect of adding the pulverized concrete into the cold-recycled mix, adding of 6 % of pulverized concrete seems to be already too much and it doesn’t bring the desired benefit.

The idea of milling cement and pulverized mechanical-chemically activated concrete together (50 % : 50 % ratio) instead of just mixing milled concrete with standard cement was investigated as well. It was presumed, that the mechanical activation influencing positively the milled concrete could have a positive effect on cement as well.

Figures 11 and 12 show that this procedure doesn’t bring as good effects as it was expected. In the case of stiffness modulus, the mix with this mixed additive (2F+3CFGRC) achieves slightly higher values than the mix with 3 % of pulverized concrete (2F+3PC) but in the case of ITS, the results are very similar. Considering that the mix 2F+3CFGRC contains 1.5 % of cement and it is therefore more expensive and less environmentally friendly, the positive effect is not big enough.
4.4 STABILIZED GRANULA MIXTURES WITH MICRO-MILLED MATERIALS

The assessment of stabilized mixes was based on the requirements of [EN 14227-1]. Therefore, the compressive strength after 28 days of specimen curing and the Rc after the freezing cycles was measured instead of indirect tensile strength typical for cold recycled mixtures as presented above (table 4 and figure 13).

Table 4: Strength characteristics of tested stabilized mixes

<table>
<thead>
<tr>
<th>Mix</th>
<th>Compressive Strength 28Rc (MPa)</th>
<th>Resistance to Water and Frost Rc (MPa) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V1</td>
<td>7.20</td>
<td>6.14 (85.24)</td>
</tr>
<tr>
<td>V2</td>
<td>4.62</td>
<td>2.86 (61.99)</td>
</tr>
<tr>
<td>V3</td>
<td>3.09</td>
<td>1.82 (58.98)</td>
</tr>
<tr>
<td>V4</td>
<td>4.03</td>
<td>2.82 (70.00)</td>
</tr>
<tr>
<td>V5</td>
<td>3.64</td>
<td>3.50 (96.19)</td>
</tr>
</tbody>
</table>

The observed characteristics showed the inappropriateness of the chosen substitution for stabilized mixtures. The substitution of standard hydraulic binders by the micro-milled recycled concrete resulted in a decrease in strength characteristics. For all investigated mixtures with pulverized concrete (V2, V3, V4) a significantly lower strength was measured after 28 days, and none of the considered alternatives complied with the requirements on Rc after freezing cycles, which should be higher than 85 % of Rc after 28 days of specimen curing.

In the case of recycled concrete milled together with limestone (V5), the mixture was compliant with the requirements on frost-heave resistance, however, it achieved significantly lower strength parameters compared to the reference mixture (V1) and therefore it can’t be considered as a full replacement.

On the contrary to the cold recycled mixtures, the addition of micro-milled (mechanical-chemically activated) recycled materials doesn’t cause any other than environmental benefits, and therefore the investigated substitution of standard hydraulic binders has no functional or economic justification.

5 CONCLUSION

The substitution of cement by recycled mechanically activated recycled concrete (FGRC) was repeatedly proven to be very beneficial in cold recycled mixtures. The pulverized concrete substitutes well the most important role of cement in a cold-recycled mix – providing very good moisture susceptibility. Moreover, although all the mixtures with pulverized concrete met the requirements of [TP 208] on sufficient strength, the growth of strength isn’t too rapid as in mixes with cement and therefore the problem with origin of hydration cracks is eliminated. Together with the economic and environmental benefits the addition of pulverized concrete into the cold-recycled mixtures is highly recommendable. Together with the results presented earlier where it was repeatedly proven that mixes with 5 % of...
pulverized concrete achieve better results than mixes with 3 %, it is recommended to add 4 – 5 % of pulverized concrete into the cold-recycled mixtures. Higher content of mechanically activated concrete causes decrease of strength. The production of the blended additive consisting of 50 % of cement and 50 % of recycled concrete milled together isn’t recommendable, because its positive effect is negligible. Adding of micro-milled limestone into the cold recycled mixes as well as adding of micro-milled materials into the stabilized mixes didn’t show any benefits.

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REFERENCES


Laboratory and Field Investigation of Pervious Concrete Pavement: A Smart City Initiative

PAPER TITLE
Laboratory and Field Investigation of Pervious Concrete Pavement: A Smart City Initiative

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KEYWORDS
1. Pervious Concrete
2. Pervious Concrete Pavement Construction
3. Field Investigation
4. Laboratory Investigation

ABSTRACT:
Pervious concrete pavements (PCPs) are class of rigid pavements that possess the ability to recharge ground water and reduce the runoff related issues in the urban areas. The increased porosity (15-35%) that comes from the reduction or elimination of fine aggregates in PC differentiates it from other pavement types. A host of laboratory studies have been performed on PC in various aspects. However, the implementation of laboratory studied PC mixtures to in-service PCPs have been least documented that help monitor their performance. Studying the laboratory and field performance will help in understanding the construction difficulties and suggest suitable remedies, with differences that would arise between laboratory and field mixtures, and long-term monitoring would further help in the selection of best performing mixtures. The objective of this study was to characterize strength and permeability properties of different PC mixtures and construction of PCP test sections using laboratory based PC mixtures for their long-term performance. The scope included: laboratory investigation of eighteen PC mixtures and construction of a 60-m PCP test sections encompassing 150 mm thick eighteen slabs of size 3 m x 2 m with embedded thermocouples that were placed on 75 mm thick sand bed. The laboratory-prepared PC mixtures had compressive strength and permeability in the range of 5-26 MPa and 0.1 to 4 cm/s, respectively. Field PCP systems consisted of subgrade with average California Bearing Ratio of 7.8% compacted to 97.94% Maximum Dry Density. The study found that on average, porosity and density of field mixtures differed by about 17.85 and 8.78%, respectively, compared to laboratory produced mixtures.
Pervious concrete (PC) is a sustainable material with interconnected pore structure that allows efficient heat and mass transfer, which is one of the solutions for many issues related to high impact development (ACI 522R-10; Chandrappa and Biligiri, 2016a). The interconnected pore structure is obtained by eliminating or reducing the fine aggregates in the aggregate gradation skeleton that also increases the volumetric porosity (Ghafoori and Dutta, 1995). Owing to the increased porosity, which lies in the range of 15-35%, the strength properties are compromised with increased permeability, thus, reducing the stormwater runoff and recharging ground water. The lower strength characteristics render PC suitable for low volume road applications. The compressive strength of PC varies in the range of 3-26 MPa, with permeability in the realm of 0.1 to 5 cm/s (Meininger, 1988; Ghafoori and Dutta, 1995; Deo and Neithalath, 2010; Ibrahim et al, 2014; Chandrappa and Biligiri, 2016b).

In PC, various mechanical and permeability properties are largely dependent upon different mixture parameters such as cement-to-aggregate ratio (c/a ratio), water-to-cement ratio (w/c ratio), gradation, aggregate size, and admixtures with c/a ratio being a significantly affecting parameter (Jimma and Rangaraju, 2015; Chandrappa and Biligiri, 2016b). Various researchers reported studies on design parameters such as flexural strength and fatigue life in addition to the basic properties. Flexural strength of PC varies in the range of 0.43 to 6 MPa, which can be significantly increased by the addition of additives such as silica fume, polymers, and admixtures such as superplasticizers (Yang and Jiang, 2003; Ibrahim et al, 2014). Fatigue behavior was investigated in few studies in which the effects of aggregate size, additives, and stress levels were quantified. It was found that with increase in the aggregate size and stress levels, the fatigue life reduced, while it was found to be improved by the addition of polymers and silica fume (Chen et al, 2013; Zhou et al, 2016). However, when the compressive fatigue of PC was evaluated, it was found that the effect of polymer addition was significant only at higher stress levels that are normally not found in the field conditions (Pindado et al, 1999).

In addition to several laboratory studies, field investigations have been carried out to understand the structural and hydrological performance of pervious concrete pavements (PCPs). The structural behavior of PCPs was investigated using falling weight deflectometer (FWD) in which the flexural stiffness was found to be in the range of 8000-17000 MPa (Vancura et al, 2011; Suleiman et al, 2011; Gogo-Abite et al, 2014). Further, Shu et al (2011)
compared the properties of field cores of PCPs and laboratory compacted field mixtures using rodding method. Further, the field cores had higher porosity, lower density, and strength magnitudes compared to laboratory prepared specimens with the same mixtures. The lower density in the PCPs was due to compaction using manual rollers. Radlinska et al. 2012 investigated an eight-year old PCP that served as a pedestrian walkway. Early deteriorations of PCP, irregular compaction, variations in the density, and irregular pore distributions were attributed to the compaction using manual rollers. The study also suggested that a single-pass of weighted rollers should be practiced to achieve better performance (Radlinska et al, 2012). PC overlay was placed on the Portland cement concrete pavement using roller screeds after extensive laboratory studies. The pavement condition survey indicated mainly five types of distresses, including: joint deterioration, stretch marks, surface sealed, de-bonding, and low-density areas (Schaefer and Kevern, 2011).

The aforementioned studies indicated that extensive laboratory investigations were carried out on PC with limited research on field investigations that discuss about the construction aspects. Moreover, in most cases either only field investigations or laboratory studies are performed without giving due importance to laboratory-and-field correlations. In addition, it has been a convention to compact PC using manual rollers that has resulted in low densities as reported in the past. Hence, there is a need to assess the applicability of other compaction methods in construction of PCPs and quantify the differences in properties of the mixtures prepared in the laboratory using standard compaction methods and those mixtures placed in the field. With these as research lacunae, the objective of this study was to undertake laboratory and field investigations on PC to understand several differences between PC properties that are bound to arise between laboratory and field mixtures’ performance. The scope of the study encompassed:

1. Design and development of eighteen PC mixtures
2. Preparation of PC cylinders
3. Determination of density and porosity (ASTM C1754)
4. Determination of permeability (Falling head permeameter)
5. Determination of compressive strength
6. Construction of PC field test sections
7. Comparison of laboratory and field produced mixtures

2. MATERIALS AND METHODS
2.1 Materials
Crushed and calcareous type aggregates of size ranging from 19 to 4.75 mm were used. The aggregates were divided into four size fractions that included: 19-13.2 mm, 13.2-9.5 mm, 9.5-6.7 mm, and 6.7-4.75 mm. Using these four size ranges, six gradations were developed that covered three single sized (P1, P2, and P3) and three combination of different sizes (P4, P5, and P6). In addition, three levels of w/c ratio and three levels of c/a ratio were selected in the preparation of the PC cylinders. The details of mixture variables are shown in Table 1. The coefficient of uniformity (Cu) was kept well below 2.0 to maintain sufficient open-graded structure even after compaction. Based on these mixture variables, eighteen mixtures were designed from Taguchi method of optimization that resulted in fifty-four specimens (three specimens per mixture type). The mix design was carried out using absolute volume mix design assuming 20% air voids. The proportions of different constituent materials in different mixtures are shown in Table 2.
### Table 1. Details of Pervious Concrete Mixture Variables

<table>
<thead>
<tr>
<th>Gradation ID</th>
<th>Aggregate size, mm</th>
<th>Gradation parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>4.75</td>
<td>6.7</td>
</tr>
<tr>
<td>P1 (Single)</td>
<td>100</td>
<td>0</td>
</tr>
<tr>
<td>P2 (Single)</td>
<td>0</td>
<td>100</td>
</tr>
<tr>
<td>P3 (Single)</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>P4 (Binary)</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>P5 (Ternary)</td>
<td>25</td>
<td>25</td>
</tr>
<tr>
<td>P6 (Quaternary)</td>
<td>25</td>
<td>25</td>
</tr>
</tbody>
</table>

w/c ratio (by wt.) | 0.25, 0.30, and 0.35

- $D_{60}$: size below, which 60% of particles are smaller than this size,
- $D_{10}$: size below, which 10% of particles are smaller than this size,
- $C_u$: coefficient of uniformity, $D_{60}/D_{10}$

### Table 2. Mixture proportions for PC Specimens

<table>
<thead>
<tr>
<th>Gradation</th>
<th>w/c ratio</th>
<th>c/a ratio</th>
<th>Aggregate, kg/m$^3$</th>
<th>Cement, kg/m$^3$</th>
<th>Water, kg/m$^3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>0.25</td>
<td>0.33</td>
<td>1462</td>
<td>487</td>
<td>122</td>
</tr>
<tr>
<td>P1</td>
<td>0.30</td>
<td>0.25</td>
<td>1562</td>
<td>390</td>
<td>117</td>
</tr>
<tr>
<td>P1</td>
<td>0.35</td>
<td>0.20</td>
<td>1628</td>
<td>326</td>
<td>114</td>
</tr>
<tr>
<td>P2</td>
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<td>120</td>
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<td>P2</td>
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<td>386</td>
<td>116</td>
</tr>
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<td>0.20</td>
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<td>321</td>
<td>112</td>
</tr>
<tr>
<td>P3</td>
<td>0.25</td>
<td>0.25</td>
<td>1599</td>
<td>400</td>
<td>100</td>
</tr>
<tr>
<td>P3</td>
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<td>0.20</td>
<td>1660</td>
<td>332</td>
<td>100</td>
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<td>P3</td>
<td>0.35</td>
<td>0.33</td>
<td>1377</td>
<td>459</td>
<td>161</td>
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<tr>
<td>P4</td>
<td>0.25</td>
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<td>1686</td>
<td>337</td>
<td>84</td>
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<tr>
<td>P4</td>
<td>0.30</td>
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<td>470</td>
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</tr>
<tr>
<td>P4</td>
<td>0.35</td>
<td>0.25</td>
<td>1515</td>
<td>379</td>
<td>133</td>
</tr>
<tr>
<td>P5</td>
<td>0.25</td>
<td>0.25</td>
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<td>399</td>
<td>100</td>
</tr>
<tr>
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<td>0.20</td>
<td>1655</td>
<td>331</td>
<td>99</td>
</tr>
<tr>
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<td>0.35</td>
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<td>1374</td>
<td>458</td>
<td>160</td>
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<tr>
<td>P6</td>
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<td>0.20</td>
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<td>338</td>
<td>85</td>
</tr>
<tr>
<td>P6</td>
<td>0.30</td>
<td>0.33</td>
<td>1415</td>
<td>472</td>
<td>142</td>
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<tr>
<td>P6</td>
<td>0.35</td>
<td>0.25</td>
<td>1520</td>
<td>380</td>
<td>133</td>
</tr>
</tbody>
</table>

#### 2.2 Methods

##### 2.2.1 Specimen Preparation

PC cylinders of size 100 mm diameter and 200 mm height were prepared using standard Proctor hammer. The fresh mixture was filled in two layers with twenty blows per layer using 2.5 kg Standard Proctor hammer (ASTM C1688). The compacted specimens were left undisturbed for 24 h and then placed in curing tank for 28 days.

##### 2.2.2 Density and Porosity

Hardened density and porosity of the PC specimens were determined as per ASTM C1754.

##### 2.2.3 Permeability

Permeability of the PC specimens was determined using an in-house fabricated falling head permeameter as reported in Chandrappa and Biligiri (2016b). The 1-Dimensional flow of water was maintained by wrapping the PC specimens with a duct tape. The time taken for the
flow of water from an initial head of 49 cm to a final head of 3 cm was recorded. The permeability was determined using Equation (1).

\[ K = \frac{aL}{At} \ln \left( \frac{h_1}{h_2} \right) \]  

(1)

2.2.4 Compressive Strength
Compressive strength of PC was determined using 3000-kN compressive strength testing machine on sulfur capped specimens. The testing was performed in controlled displacement mode with displacement rate of 1 mm/min.

3. LABORATORY RESULTS
Eighteen mixtures encompassing fifty-four specimens considering three replicates per mixture were tested in the laboratory. The relation between various properties of PC is shown in Figure 3 and the individual magnitudes of different properties are shown in Table 3. More elaborate discussions of the laboratory investigations can be found elsewhere in Chandrappa and Bilgiri (2016b, 2017a, 2017b).

Figure 3. Relations between different PC Properties investigated in the Laboratory
<table>
<thead>
<tr>
<th>Mixture</th>
<th>Density, kg/m³</th>
<th>Porosity, %</th>
<th>UCS, MPa</th>
<th>Permeability, cm/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1-0.35-0.20</td>
<td>1802.27</td>
<td>35.99</td>
<td>10.26</td>
<td>1.85</td>
</tr>
<tr>
<td>1790.17</td>
<td>37.20</td>
<td>8.98</td>
<td>2.00</td>
<td></td>
</tr>
<tr>
<td>1782.54</td>
<td>37.07</td>
<td>11.24</td>
<td>1.67</td>
<td></td>
</tr>
<tr>
<td>P1-0.30-0.25</td>
<td>1839.19</td>
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<td>12.92</td>
<td>1.61</td>
</tr>
<tr>
<td>1867.84</td>
<td>32.71</td>
<td>16.85</td>
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<td></td>
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<td>1873.57</td>
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<tr>
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<td>1835.37</td>
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</tr>
<tr>
<td>1836.65</td>
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<td>1.31</td>
<td></td>
</tr>
<tr>
<td>1847.47</td>
<td>33.09</td>
<td>11.40</td>
<td>1.19</td>
<td></td>
</tr>
<tr>
<td>P2-0.35-0.20</td>
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<td>1.20</td>
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<td>1836.65</td>
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<td></td>
</tr>
<tr>
<td>1847.47</td>
<td>33.09</td>
<td>11.40</td>
<td>1.19</td>
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<td>1.31</td>
</tr>
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<td>9.82</td>
<td>1.36</td>
<td></td>
</tr>
<tr>
<td>1865.30</td>
<td>33.03</td>
<td>12.16</td>
<td>1.69</td>
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<tr>
<td>P2-0.25-0.33</td>
<td>1850.02</td>
<td>33.47</td>
<td>9.93</td>
<td>1.34</td>
</tr>
<tr>
<td>1909.86</td>
<td>31.37</td>
<td>17.50</td>
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</tr>
<tr>
<td>1887.58</td>
<td>32.07</td>
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<td>1.39</td>
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</tr>
<tr>
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<td>34.59</td>
<td>9.78</td>
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<td>33.03</td>
<td>12.16</td>
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</tr>
<tr>
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<tr>
<td>1827.74</td>
<td>35.32</td>
<td>8.44</td>
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<tr>
<td>1855.75</td>
<td>34.24</td>
<td>8.22</td>
<td>2.21</td>
<td></td>
</tr>
<tr>
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<td>2174.37</td>
<td>35.70</td>
<td>9.75</td>
<td>2.17</td>
</tr>
<tr>
<td>2164.83</td>
<td>35.32</td>
<td>8.44</td>
<td>2.62</td>
<td></td>
</tr>
<tr>
<td>2137.45</td>
<td>34.24</td>
<td>8.22</td>
<td>2.21</td>
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</tr>
<tr>
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<td>37.61</td>
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</tr>
<tr>
<td>1768.53</td>
<td>37.23</td>
<td>5.66</td>
<td>2.00</td>
<td></td>
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<tr>
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<td>37.61</td>
<td>6.63</td>
<td>2.04</td>
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<tr>
<td>2084.29</td>
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<td>27.55</td>
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<td>1967.79</td>
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<td>0.73</td>
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<td>2009.17</td>
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<td>17.84</td>
<td>0.71</td>
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<td>P5-0.30-0.20</td>
<td>1913.68</td>
<td>32.65</td>
<td>13.34</td>
<td>1.66</td>
</tr>
<tr>
<td>1920.68</td>
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<td>11.88</td>
<td>1.84</td>
<td></td>
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<tr>
<td>1923.86</td>
<td>31.95</td>
<td>15.13</td>
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<tr>
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<td>1862.75</td>
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<td>10.94</td>
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<td>2249.18</td>
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<td>25.64</td>
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<tr>
<td>P6-0.30-0.33</td>
<td>2147.32</td>
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<td>2184.88</td>
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<td>24.93</td>
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<tr>
<td>2161.96</td>
<td>17.24</td>
<td>24.54</td>
<td>0.31</td>
<td></td>
</tr>
<tr>
<td>P6-0.35-0.25</td>
<td>2079.60</td>
<td>22.08</td>
<td>19.96</td>
<td>0.39</td>
</tr>
<tr>
<td>2075.60</td>
<td>22.33</td>
<td>19.25</td>
<td>0.41</td>
<td></td>
</tr>
<tr>
<td>2072.30</td>
<td>21.89</td>
<td>15.39</td>
<td>0.38</td>
<td></td>
</tr>
<tr>
<td>P6-0.25-0.20</td>
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<td>34.17</td>
<td>10.67</td>
<td>2.06</td>
</tr>
<tr>
<td>1874.21</td>
<td>33.86</td>
<td>9.77</td>
<td>2.17</td>
<td></td>
</tr>
<tr>
<td>1869.75</td>
<td>33.86</td>
<td>10.24</td>
<td>1.99</td>
<td></td>
</tr>
</tbody>
</table>
Among these eighteen mixtures, six mixtures were selected as field test sections mainly to study the effect of aggregate gradation, whose laboratory results are summarized in Table 4.

<table>
<thead>
<tr>
<th>Field Mixtures</th>
<th>Laboratory Results*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hardened Density, kg/m³</td>
</tr>
<tr>
<td>P2-0.30-0.25</td>
<td>1869.54</td>
</tr>
<tr>
<td>P4-0.35-0.25</td>
<td>1989.44</td>
</tr>
<tr>
<td>P4-0.35-0.20</td>
<td>1968.33</td>
</tr>
<tr>
<td>P3-0.35-0.25</td>
<td>2055.66</td>
</tr>
<tr>
<td>P6-0.35-0.25</td>
<td>2075.83</td>
</tr>
<tr>
<td>P5-0.35-0.20</td>
<td>2019</td>
</tr>
</tbody>
</table>

*Few mixtures were independently investigated for only flexural strength

4. CONSTRUCTION OF PERVIOUS CONCRETE PAVEMENT PILOT TEST SECTIONS

4.1 Site Selection
The site for the construction of PCP pilot test sections was selected in the Kendriya Vidyalaya school premises on-campus Indian Institute of Technology Kharagpur (IIT-KGP). The Google map view of the site selected is shown in Figure 4. The white patch on the Figure indicates the section, which was 60-meter long. The site before the commencement of construction of the test section is shown in Figure 5.

![Figure 4. Google Map View of Site of PCP Pilot Test Section](image)

![Figure 5. Site before Commencement of Construction](image)

4.2 Site Preparation
The topsoil in the site consisted of loose fill material, and therefore it was excavated to a depth of 16” to obtain natural soil subgrade. The natural soil subgrade was further compacted using 1-tonne manual roller until a flat surface was obtained. No efforts were made to attain
any particular density of the natural soil while compacting. The soil samples were collected from the site to determine engineering properties. In the laboratory, the Proctor density, optimum moisture content, and California Bearing Ratio (CBR) was determined in accordance with (IS 2720). In the field, sand replacement method was used to determine the field density of the natural soil subgrade. Figure 6 shows series of photographs depicting the steps followed during the soil compaction process.

![Figure 6. Compaction of Soil in Site](image)

(a) and (b) Compaction of Soil Using Manual Roller; (c) View of Finished Surface

4.3 Pervious Concrete Pavement System (PCPS)

4.3.1 Preparation of Sand Layer

PCPS was designed to have three layers that consisted of 3 m * 2 m * 0.15 m deep PC slab, 75 mm fine sand layer (300-150 microns), and natural subgrade. The sand layer was spread and compacted using a float to obtain a flat surface. The purpose of sand layer was to provide uniform surface for the PC slabs and also to function as a temporary storage for stormwater. Aggregate base was not provided since the PCPS at the present site was anticipated to serve only the non-motorized traffic. Figure 7 shows different operations in the preparation of sand layer.

![Figure 7. Different Operations in Application of the Sand Layer](image)

(a) Spreading; (b) Wetting; (c) Finishing and Leveling; (d) Finished Surface

4.3.2 Construction of PC slabs

Amongst the eighteen mixtures that were tested in the laboratory, six mixtures were selected and placed as field PC test sections. The object of the selection of different mixtures was mainly to understand the effect of different gradations. In addition, mixtures with low w/c ratio and c/a ratio were avoided as the laboratory study showed that such mixtures were too harsh and were easily prone to abrasion. The details of the mixtures selected for the
The steps involved in the construction of PC slabs comprised of: wetting the sand layer, mixing the constituent materials, transportation and placing, compaction using plate vibrator, spraying white curing compound, and covering the fresh compacted slab with a curing compound. Vibratory method was utilized since many previous studies showed that compaction using manual rollers would lead to low density, high porosity, and lower strength properties (Shu et al, 2011; Radlinska et al, 2012). Photographs displaying the steps followed in the construction of PC slabs are shown in Figure 8.

Table 5. Details of Mixtures used in PC Slab Construction

<table>
<thead>
<tr>
<th>Mixtures</th>
<th>Aggregate, kg</th>
<th>Cement, kg</th>
<th>Water, kg</th>
<th>Admixture, kg</th>
</tr>
</thead>
<tbody>
<tr>
<td>P2-0.30-0.25</td>
<td>0</td>
<td>0</td>
<td>1660</td>
<td>0</td>
</tr>
<tr>
<td>P4-0.35-0.25</td>
<td>0</td>
<td>0</td>
<td>822</td>
<td>822</td>
</tr>
<tr>
<td>P4-0.35-0.20</td>
<td>0</td>
<td>0</td>
<td>866</td>
<td>866</td>
</tr>
<tr>
<td>P3-0.35-0.25</td>
<td>0</td>
<td>1645</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>P6-0.35-0.25</td>
<td>411</td>
<td>411</td>
<td>411</td>
<td>411</td>
</tr>
<tr>
<td>P5-0.35-0.20</td>
<td>0</td>
<td>866</td>
<td>433</td>
<td>433</td>
</tr>
</tbody>
</table>

The air temperature during the period of construction was > 38 °C that made the mixtures at the surface look dry, which appeared to create problems during compaction. To avoid such problems, water was sprayed on the fresh slab prior to compaction to increase the wetness at the surface. In addition, the mixture was placed 1-inch above the formwork, which was later compacted in flush with formwork to ensure sufficient compaction. Three cylinders per slab
were prepared from the field mixtures to check the reproducibility and repeatability between the slabs of the same mixture and compare those with the laboratory results. The view of the completed test sections is shown in Figure 9.

Figure 9. (a) Final View of the Test Sections; (b) Close-up View of the Surface Texture of P6 Graded Slab

5. FIELD INVESTIGATION AND COMPARISON OF LABORATORY-FIELD SPECIMENS

5.1 Soil Investigation

Soil investigations were performed to determine the engineering performance of PCPSs. The average results (three locations) of the different soil properties are shown in Table 6. The numbers in the parentheses indicate one standard deviation.

<table>
<thead>
<tr>
<th>Table 6. Details of Soil Conditions in Test Sections</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average bulk density of sand, g/cc</td>
</tr>
<tr>
<td>Average field density of soil, g/cc</td>
</tr>
<tr>
<td>Average maximum dry density, g/cc</td>
</tr>
<tr>
<td>Average OMC, %</td>
</tr>
<tr>
<td>Average CBR, %</td>
</tr>
<tr>
<td>Percent of MDD achieved, %</td>
</tr>
</tbody>
</table>

The average field density was found to be 1.952 g/cc with low standard deviation indicating uniform compaction. Moreover, the soil at the test site was compacted to 97.94% of maximum dry density (MDD), although no efforts were made to control compaction to achieve any particular field density. The average CBR was 7.82%, which was greater than the minimum required for low-volume rigid pavements as per IRC: SP: 62.

5.2 Comparison of Laboratory and Field Mixtures

The density and porosity of laboratory compacted (LC) and field produced-laboratory compacted mixtures (FLC) were compared. This comparison was mainly performed to understand and quantify the differences that would arise between laboratory and field produced mixtures. The percentage change in porosity and density in FLC compared to (LC) mixtures is shown in Figure 10. It can be seen that in most cases FLC had higher porosity and lower density compared to LC mixtures, in which a maximum difference of 40% in porosity and 15% in density was found. Shu et al (2011) reported similar kind of observation, albeit the percentage changes in the magnitudes of porosity and density were not reported. On an average, percentage difference in porosity and density was found to be 18.0 and -7.44%, respectively. Even though the same aggregate type was used for laboratory and field mixtures, the differences in porosity and density could have arisen due to the factors such as efficiency of the concrete mixer in homogeneously distributing the constituent materials, environmental conditions, and loading capacity. These differences in the properties were helpful in determining suitable shift factors, which could avoid overestimation of properties of the field-placed PCPs based on the laboratory results. Moreover, field cores were not
studied as it was found that during coring; the aggregate particles separated / broke down as shown in Figure 11, which could have resulted in lower accuracy of the magnitudes of PC properties.

Figure 10. Comparison of Laboratory and Field Mixture Properties
(a) Porosity; (b) Density

Figure 11. Coring in PCP
6. Conclusions

The objective of the study was to investigate properties of pervious concrete mixtures and construct pervious concrete pavement test sections to understand laboratory-to-field correlations. Based on the laboratory and field studies, the following conclusions were made:

- Laboratory investigation depicted excellent relations between different properties of PC, which were rational.
- The placing of materials for one slab took around 45-60 minutes, which made the surface look dry. To avoid loose mix on the surface, water was sprayed before compacting and finishing.
- Compaction using plate vibration seemed to be rational that provided sufficient density without creating any wet spots (clogged with the cement paste). This technique could be used elsewhere instead of manual rollers, which also resulted in lower densities. Moreover, vibratory compaction reduced efforts involved in finishing the surface.
- Differences in laboratory and field-produced mixtures were quantified in terms of percentage change in porosity and density. Field mixtures had 18% higher porosity and -7.44% lower density compared to laboratory produced mixtures.
- It is envisioned that accounting for the differences between field produced and laboratory-produced mixtures would assist in rational design of PCPs in future.

ACKNOWLEDGEMENTS

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Senegal Roads Case Study: Improving delivery and economic returns of roads projects

**ABSTRACT:**

The Millennium Challenge Corporation (MCC), a US based foreign assistance donor, focusing on economic development through economic growth. The MCC is also actively involved in introducing new technologies and innovative approaches to host countries. MCC worked with the Government of Senegal to design and rehabilitate the National Road 2 (RN2) in Northern Senegal from 2010 to 2015.

The paper outlines the overall RN2 project, responsible entities and the major challenges and hurdles faced by the project team throughout the project lifecycle. One of the major challenges in Senegal and particularly on this road segment, was the availability of suitable borrow materials for the rehabilitation of the road segment within the project zone.

The paper describes the design review process that was conducted to update the project design during the construction phase in order to reduce life cycle environmental and economic costs through the recycling of the existing pavement. The paper also focuses on the introduction of a performance based specification for the IRI that was used to incentivize the contractor to deliver a higher quality investment for the road users.

Particular focus will be on the design review process used during the construction phase and the final results on the overall project execution, including results from adopting a lower IRI threshold.

The paper concludes with lessons learned from the project, which will be used in further evaluation of MCC transport related projects.
MCC Senegal Roads Case Study: Improving Delivery and Economic Returns of Road Projects

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1 INTRODUCTION

The Millennium Challenge Corporation (MCC) is a small, U.S. Government agency established in 2004 with an innovative and tested approach to fighting global poverty. The MCC model focuses on economic growth and helping people lift themselves out of poverty through projects like power, clean water, land rights and roads — delivering large infrastructure investments coupled with support for prudent policy and institutional reforms. As part of this mission, MCC strives to transfer improved technology and project management approaches to build capacity in developing nations.

The objective of this paper is to highlight transport sector innovation on a recent MCC-funded road investment in Senegal. This experience illustrates several important aspects of MCC’s focus on improving economic returns on its investments throughout the project lifecycle. Secondly, this paper presents lessons and findings of a 2015 internal review of the agency’s road projects undertaken to improve future performance.

2 BACKGROUND

In September 2009, MCC and the Government of the Republic of Senegal (GoS) signed a $540 million compact aimed at reducing poverty and increasing economic growth by unlocking the country’s agricultural productivity and expanding access to markets and services. The two primary compact projects, Roads Rehabilitation and Irrigation and Water Resource Management, strategically invested in the road network and essential irrigation schemes in the Senegal River Valley in the north and the Casamance region in the south. The compact priorities were identified to align with the country’s comprehensive long-term objectives of enhancing economic growth and food security.

The Millennium Challenge Account-Senegal (MCA-S) was the GoS 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.

A primary activity of the Roads Rehabilitation Project was the National Road 2, or Route Nationale 2 (RN2). The RN2 is one of the longest national road links in Senegal, stretching from the outskirts of the capital, Dakar, northbound to Saint Louis along the border with Mauritania, to the far eastern border with Mali. This investment was designed to advance economic growth in the Senegal River Valley, a zone known for its potential as a producer of rice. As part of the compact, which was completed in 2015, MCC financed a 122 km rehabilitation of an existing segment of the RN2 from Richard Toll to Ndioum, including a 150 meter bridge over the Senegal River, as shown in Figure 1 below.
A key finding of MCC’s internal review of road projects was that road investments should undergo an enhanced design review process throughout the project lifecycle (Patel et al, 2017). In the case of Senegal, the design review process extended into the construction phase, where efforts were made to further reduce environmental and economic costs. The following section outlines how this improved design review process worked in practice.

3 DESIGN REVIEW AND OPTIMIZATION

Access to quality materials is a continual challenge in road infrastructure works in West Africa. In the case of Senegal, the supply of quality materials for road construction is becoming more limited, which can have costly impacts for future generations. As part of the MCC approach in Senegal to minimize economic and environmental costs, the engineering and environmental teams worked to reduce, where possible, suboptimal use of materials and focus on maximizing existing materials that are readily available. This effort continued through different stages of the project lifecycle, from design to construction.

A. Borrow Materials Were Scarce: A critical issue identified on the RN2 during due diligence was a lack of sufficient quantities of suitable borrow materials (sand and gravel) to rehabilitate and widen the road. MCC due diligence consultants estimated a need of up to 850,000 m$^3$ of borrow materials for the project. However, there were only 65,000 m$^3$ of known suitable materials in the project zone, and the additional haulage cost from other known sources threatened major cost overruns.

Additional investigations were undertaken during the design stage and approximately 20 additional borrow areas were discovered along the project length with sufficient capacity to supply the project. Further effort was added during the early stages of the contractor mobilization period to identify more borrow sites, which resulted in several more borrow sites along the project itinerary and helped reduce overall transport haulage costs. Overall, the final haulage costs were reduced from the engineers’ estimate by 29% for the pavement platform and 36% for structure materials (subbase and base).

However, the greatest savings came with the design review and optimization that was requested by MCC and MCA-S just after construction began.

B. Design Optimization during the Construction Phase: In French-speaking West Africa, an engineer typically undertakes a limited amount of geotechnical testing to determine the most optimal design, while the contractor is responsible for conducting a much larger geotechnical campaign to finalize the design for construction. Initial geotechnical investigations undertaken by the contractor for this project indicated much better characteristics than initially anticipated. As a result, MCC and MCA requested the contractor complete additional geotechnical investigations along the entire project alignment and provide an additional design optimization, specifically a recycling of the existing structure into the subbase and/or base for those sections showing better results. This design optimization also included an improved
balance of the cut and fills on the project to reduce haulage quantities even further. Overall quantity savings are presented in Table 1.

Table 1: Overall quantity savings for the RN2 Road

<table>
<thead>
<tr>
<th></th>
<th>Engineers Design</th>
<th>Final Executed Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cuts</td>
<td>122,826 m³³</td>
<td>104,323 m³³</td>
</tr>
<tr>
<td>Fills</td>
<td>211,439 m³³</td>
<td>177,419 m³³</td>
</tr>
<tr>
<td>Subbase</td>
<td>234,550 m³³</td>
<td>207,836 m³³</td>
</tr>
</tbody>
</table>

Source: SGS, 2015a

C. Results

Cost control on construction projects is challenging, yet essential for maintaining the expected economic benefits of the project investment. Chong and Hopkins (2016) found that, for projects authorized from 2005 and 2010, MCC transport project cost increases of more than 135% from initial funding authorization through the project lifecycle were similar to its peer group at the World Bank. Chong and Hopkins also noted that the MCC cost increases were generally accompanied by a 33% reduction in project scope as well.

In comparison to these findings, the Senegal RN2 Project may serve as a point on the hypothetical efficiency frontier described by Chong and Hopkins. Table 2 below shows the progression of the RN2 activity costs over time using the same terminology employed by Chong and Hopkins (2016). The RN2 activity costs decreased by more than 20% over the entire project lifecycle.

Table 2: Progression of costs for the RN2 activity

<table>
<thead>
<tr>
<th>Element</th>
<th>Funding Authorization (FA)</th>
<th>Engineers Estimate (EE)</th>
<th>Contract Award (CA)</th>
<th>Final Cost (FC) for Initial Scope</th>
<th>Final Cost with Additional Scope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Construction</td>
<td>$ 86,100,000</td>
<td>$ 76,200,000</td>
<td>$ 64,900,000</td>
<td>$60,700,000</td>
<td>$ 65,800,000</td>
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<tr>
<td>RN2 Road (121.6km)</td>
<td>$ 73,100,000</td>
<td>$ 63,300,000</td>
<td>$ 52,000,000</td>
<td>$47,900,000</td>
<td>$ 53,000,000</td>
</tr>
<tr>
<td>Ndioum Bridge (150meters)</td>
<td>$ 13,000,000</td>
<td>$ 12,900,000</td>
<td>$ 12,900,000</td>
<td>$12,800,000</td>
<td>$ 12,800,000</td>
</tr>
<tr>
<td>Design/Supervision</td>
<td>$ 8,200,000</td>
<td>$ 5,400,000</td>
<td>$ 5,400,000</td>
<td>$4,000,000</td>
<td>$ 5,400,000</td>
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<tr>
<td>RN2 Road (121.6km)</td>
<td>$ 8,200,000</td>
<td>$ 2,700,000</td>
<td>$ 2,700,000</td>
<td>$2,700,000</td>
<td>$ 2,700,000</td>
</tr>
<tr>
<td>Ndioum Bridge (150meters)</td>
<td>$ 8,200,000</td>
<td>$ 2,700,000</td>
<td>$ 2,700,000</td>
<td>$2,700,000</td>
<td>$ 2,700,000</td>
</tr>
<tr>
<td>Resettlement</td>
<td>$ 80,000</td>
<td>$ 1,069,000</td>
<td>$ 838,000</td>
<td>$780,000</td>
<td>$ 780,000</td>
</tr>
<tr>
<td>RN2 Road (121.6km)</td>
<td>$ 80,000</td>
<td>$ 199,000</td>
<td>$ 199,000</td>
<td>$141,000</td>
<td>$ 141,000</td>
</tr>
<tr>
<td>Ndioum Bridge (150meters)</td>
<td>$ 80,000</td>
<td>$ 870,000</td>
<td>$ 639,000</td>
<td>$639,000</td>
<td>$ 639,000</td>
</tr>
<tr>
<td>Total</td>
<td>$ 94,380,000</td>
<td>$ 82,669,000</td>
<td>$ 71,138,000</td>
<td>$66,880,000</td>
<td>$ 71,980,000</td>
</tr>
</tbody>
</table>

Source: MCC, 2017

The RN2 road project results are plotted below in Figure 2 alongside the corresponding findings from Chong and Hopkins.
The quality of the constructed road is paramount to the economic model’s robustness but also for optimal use of scarce materials. A road that degrades severely in a few years will waste and harm economic and environmental resources. As part of the overall approach in Senegal, a higher level of detailed oversight was used to ensure greater quality control during the design and construction phase.

4 FOCUS ON QUALITY

A. Integrating Empirical Experience into the Design: One of the more prevalent pavement design techniques in Senegal is to use a thin asphalt wearing course over a cement treated lateritic base course on top of a non-stabilized subbase. Heavy trucks are generally overloaded in Senegal and throughout the sub-region, which poses a significant risk to the cement treated base course, if it is designed to have too much cement or rigidity.

MCC and its independent engineering firm, along with MCA, undertook a 2,000 km survey of existing lateritic cement treated roads in Senegal and simulated the structural performance of various road structures to understand the behavior of lateritic cement treated base courses in Senegal. One of the findings of the analysis was that the most probable cause of premature failure of these roads was the use of too much cement, which turned the base course into a semi-rigid structure acting in flexure, as opposed to a flexible structure acting in compression. In other words, the base course would fail, due to its inability to absorb stress in a semi-rigid structure, as opposed to absorbing these stresses in a flexible structure.

As a direct outcome of the survey, it was agreed that a new testing approach (the Brazilian Test) would be used to provide an indirect measure of tensile strength, which is strongly correlated to flexural strength. Tests (NF EN 13286-42) at 2.0%, 2.5%, 3.0% and 3.5%, were required for each lateritic sample with a determined set of acceptable limits. The final design quantity was set at 4% to be conservative for estimating quantity purposes; the final field usage varied depending on the borrow pits, but generally ranged from 2.5 to 3% cement, which resulted in an overall cement savings of 42% or about half a million USD and a higher degree of confidence in the durability of the pavement design. By working with the local engineers, academics, road agency and other institutions, MCC helped transfer improved engineering designs and practices to the local road agency.

B. Monitoring Quality during Construction: Quality control is an essential element of construction management to ensure that the product delivered meets the requirements stated in technical specifications the expectations of stakeholders. One of the innovative MCC approaches that has been applied to several recent MCC roads projects is the development of itinerary diagrams to graphically illustrate the testing results in real time by road length along with the technical specification requirement(s). These itinerary diagrams were the basis of monthly reports and all MCC independent engineering site visits in Senegal; and as such, MCC was able to quickly and efficiently review progress on site and discuss relevant quality-related issues. These itinerary diagrams formed the foundation of MCC and MCA’s request for a design optimization.

**Figure 2:** RN2 Project per km costs relative to projected Chong and Hopkins applied findings (Source: MCC, 2017 and Chong and Hopkins, 2016)
An example is shown in Figure 3 below.

![Figure 3: Itinerary diagram showing the characteristic deflections along the RN2 by kilometer (Source: SGS, 2015b)](image)

All of the geotechnical data was collected and represented in itinerary diagram format and transferred to the road agency for use in monitoring and evaluation of the pavement structure over time. A major advantage of this approach is that if a problem specific to these geotechnical characteristics were to arise on the RN2 project section, the road agency can relate that problem to what occurred during the construction phase. This can inform future construction practices and make solving the immediate problem easier.

For MCC, some of these data points, like the deflections, course thickness, and the international roughness index, are used to update the HDM4 model for post-construction evaluation.

5 IMPROVING ECONOMIC BENEFITS TO SOCIETY WITH AN IMPROVED IRI

A high-quality, smooth road has major benefits for the road agency and users, but also for the environment. The following section outlines how MCC and MCA-S introduced an International Roughness Index (IRI) performance clause to incentivize the contractor to build a high-quality, smooth road.

The IRI is an open-ended indicator of pavement roughness based on a theoretical minimum of 0 m/km, which indicates a perfectly smooth surface. See Figure 4 for a practical description relating the scale to pavement types and conditions, and to vehicle speeds. A lower IRI indicates a smoother pavement, in addition to reduced vehicle operating costs (VOC) and travel time costs (TTC). The IRI is an important parameter in measuring the economic benefits of a road investment as higher IRI values have an exponential effect on road user costs, as well as the lifetime of a road surface investment.
Figure 4: International Roughness Index Scale (Source: Sayers et al, 1986)

Typical values in sub-Saharan Africa for post-works IRIs on a national road vary from 2.5 m/km to 4.5 m/km. (This is considerably higher than post-works values in North America, which range from 1.0 m/km to 1.7 m/km on national highways.) Annual pavement deterioration, as measured by the IRI, ranges from 0.05 m/km/year up to 0.33 m/km/year, per the following sources shown in Table 3 below. Thus, a road that begins at a lower IRI value would, theoretically, perform better over its lifetime if it begins its performance (as measured by IRI) at a lower value.

Table 3: Summary of annual estimated IRI linear deterioration rates (m/km)

<table>
<thead>
<tr>
<th>Source</th>
<th>Annual Estimated IRI Linear Deterioration Rates (m/km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HDM4 Low Traffic (Morosiuk, Riley and Odoki, 2004)</td>
<td>0.2 m/km</td>
</tr>
<tr>
<td>HDM4 High Traffic (Morosiuk, Riley and Odoki, 2004)</td>
<td>0.3 m/km</td>
</tr>
<tr>
<td>HDM III (Hunt and Bunker, 2002)</td>
<td>0.24 m/km</td>
</tr>
<tr>
<td>PDAT (Hunt and Bunker, 2002)</td>
<td>0.15 m/km</td>
</tr>
<tr>
<td>NIMPAC (Hunt and Bunker, 2002)</td>
<td>0.14 m/km</td>
</tr>
<tr>
<td>ARRB (Hunt and Bunker, 2002)</td>
<td>0.1 m/km</td>
</tr>
<tr>
<td>LTPP (Dong and Huang, 2012)</td>
<td>0.16 to 0.33 m/km</td>
</tr>
<tr>
<td>AUSTROADS (Martin, 2004)</td>
<td>0.01 to 0.05 m/km</td>
</tr>
</tbody>
</table>

MCC and MCA-S adopted a default annual deterioration value of 0.2 m/km. This value is most likely higher than actual deterioration for this particular road, because it is in an arid, desert climate. The road agency had not recorded any corrective (IRI-related) work on the road in over 15 years. As none was performed, the road’s recent pre-construction IRI value measured by the road agency was an average of 2.8 m/km. This would suggest a rate of deterioration of no more than 0.05 m/km/year, since the original surface must have had an IRI greater than zero. As-built records and any measure of IRI performance were not available to support a more objective conclusion; however, it is doubtful that HDM4 linear deterioration rates would apply, as the road would have begun with a value of less than zero.

By providing a longer pavement lifespan and smoothness, more economic benefits can be generated for society for both the road user (reduced VOC and TTC) and the road agency (reduced maintenance expenditures). In this particular case, the results show that these benefits are derived mostly from vehicle operating costs savings. Travel time savings are negligible in this case, because at these levels of IRI, pavement smoothness does not affect vehicle speeds.

A. Improving User Cost Savings: As part of the compact agreement with the Government of Senegal, a monitoring and evaluation IRI outcome target of 2.5 m/km was agreed upon, as this was the lower bound of the Government road agency’s experience with contractors.

In order to motivate the contractor to obtain the lowest IRIs economically justifiable, performance incentives were written into the pavement contract for the 50 mm asphalt resurfacing. Bonuses of 10% were added when the IRI was below 1.5 m/km and 10% penalties were subtracted when the IRI was above 2.5 m/km and below 3.5 m/km. Also, per the contract, the asphalt resurfacing was not to be paid to the contractor on 1,000 m sections where the IRI was above 3.5 m/km. The IRI was measured with the French standard (NF P 98-218-3).

The contractor invested in new equipment and training, including a new top-quality high compaction density Vögele paver with a laser guiding system, GPS controlled grading equipment, and the hiring of a pavement placement engineer from Europe to train the contractor’s paving crew in proper placement techniques. See Figure 5 below. In addition, in order to achieve this lower bound target, the contractor needed to pay close attention to the overall quality of the base course grading and pavement placement requirements. Otherwise, recouping the initial investment in new equipment would be at risk.
Figure 5: Example of the additional paving equipment purchased by the contractor: Vögele paver with a laser guiding system, GPS controlled grading equipment. (Source: Towles, 2014)

The resulting IRI measurements are shown in Figure 6 below.

Figure 6: IRI measurements along the RN2 by distance and direction relative to target IRI (Source: SGS, 2015b and LBG, 2015)

The engineer determined the following, with an average IRI of 1.4m/km:
- 82% of the 1,000 m sections had an IRI below 1.5 m/km and were subject to a 10% bonus on the asphalt layer;
- 17% of the 1,000 m sections has an IRI between 1.5 and 2.5 m/km and were subject to neither a bonus, a penalty nor a rejection; and
- 1% of the 1,000 m section had an IRI above 2.5 m/km and were subject to a penalty of 10% on the asphalt layer.

A key question is: what were the economic benefits of this incentive payment system and the resulting lower IRIs? To determine this answer, the target IRI of 2.5m/km and the achieved IRI of 1.4 m/km were both modeled using the original HDM-4 workspace for Senegal to determine the discounted present value of the VOC savings for the 1.4 m/km versus the VOC saving for the 2.5 m/km compact target value.
The 10% discounted user savings for the target 2.5 m/km work was calculated and was subtracted from the average 1.4 m/km value, which resulted in a total net present savings of just under $2 million, which resulted in a 20% improvement in the economic rate of return (ERR).

Subtracting the performance payment provided to the contractor from the estimated user cost savings, the net present value of the savings, or economic benefits to society from this performance incentive is approximately $1,000,000. In addition to the total net present value of user cost savings, the pavement should theoretically be in a much better condition after 20 years.

In addition to the total net present value of the user cost savings, the local contractor has also learned how to deliver a higher-quality pavement, which may make the local contractor more competitive on other IRI performance-based contracts.

B. Lowering Emissions

The IRI performance clause also had a direct effect on the reduction of vehicular emissions, notably hydrocarbons (HC), carbon monoxide (CO), nitrogen oxides (NOx), sulphur dioxide (SO2), carbon dioxide (CO2), particulates, and lead (Pb). Improved road performance translates directly into lower fossil fuel consumption and reduced emissions. The HDM4 software provides an estimated quantity change per emission, which is shown below in Figure 7. While this particular project did not monetize emissions reductions as part of the economic benefit, the reduction of the IRI and its effect on reduced environmental emissions may be another important justification for providing a performance-based IRI clause in the future.

<table>
<thead>
<tr>
<th>Emission</th>
<th>Reduction %</th>
</tr>
</thead>
<tbody>
<tr>
<td>HC</td>
<td>2.9%</td>
</tr>
<tr>
<td>CO</td>
<td>2.6%</td>
</tr>
<tr>
<td>NOx</td>
<td>2.2%</td>
</tr>
<tr>
<td>SO2</td>
<td>3.3%</td>
</tr>
<tr>
<td>CO2</td>
<td>2.7%</td>
</tr>
<tr>
<td>Particulates</td>
<td>3.3%</td>
</tr>
<tr>
<td>Pb</td>
<td>1.7%</td>
</tr>
</tbody>
</table>

*Figure 7: Predicted HDM4 emissions reduction from a 2.5 m/km IRI to 1.4 m/km IRI, (Source: MCC, 2017)*

6 RECOMMENDATIONS FOR FUTURE ROADS PROJECTS

First lesson: The primary lesson of this experience in Senegal is that an IRI performance-based specification can help contractors improve overall performance and sustainability of investments, while providing additional benefit to the local economies, businesses and beneficiaries through lower vehicle operating costs. Having proven that lower IRI targets are possible, future road projects in the region can apply the same approach to improve overall investment quality and performance. By using HDM4 in the design of the performance measures, MCC, MCA and national road agencies can optimize performance payments and apply this know-how to other projects to optimize road agency expenditures and improve overall road network performance, as measured by the IRI.

Second lesson: The second lesson from MCC’s experience in Senegal is that recycling, value engineering and design optimization should be an integral part of the design process. Additionally, engineers should be encouraged to: 1) maximize the reuse of existing material on the road, which often represents a better material or asset than any additional material hauled from quarries many kilometers away, even if that means management of institutional barriers with the road agency is required due to perception that recycling is second-hand use, 2) capitalize on new pavement engineering design technologies, such as new performance-based asphalt mix design methods, characterization of materials, pavement structural design, life-cycle cost analysis, local performance-based structural designs, and pavement monitoring technologies.
MCC and MCA-S requested a structural design optimization of the second half of the RN2 project. This consisted of the full reclamation of the existing road and lesser additional and unnecessary materials while resulting in equally good pavement deflection measurements, indicative that this optimization has not resulted in any weakening of the structure. The result was that construction cost per km was reduced about 16% from $440,000 USD/km to $370,000 USD/km.

**Third lesson:** The last lesson learned by MCC is the importance of managing time, cost and quality through project documentation that is easy to understand and represented in graphical formats, known as itinerary diagrams. This has shown to improve overall communication of field performance for all involved parties and keeps everyone focused on the task at hand. Going forward, this approach is being used during the initial project evaluation phase to properly evaluate road investments and serve as a solid baseline for the project. Secondly, these itinerary diagrams provide MCC with the needed information to continuously update the project ERR, as needed, with a reliable source of information and also serve as the basis for a rigorous monitoring and evaluation program.

**8 REFERENCES**


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SGS, “Presentation recapitulative du project,” May 19, 2015a.


**PAPER TITLE**: Abu Dhabi City Municipality’s Initiatives for the Application of Value Engineering in Road and Infrastructure Projects

<table>
<thead>
<tr>
<th>TRACK</th>
<th>Mohammad Najam KHAN</th>
<th>TECHNICAL ADVISOR – BRIDGES AND ROAD STRUCTURES</th>
<th>ABU DHABI MUNICIPALITY</th>
<th>UNITED ARAB EMIRATES</th>
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</thead>
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<td>Sami Abdul Qader ALHASHMI</td>
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<td>ABU DHABI CITY MUNICIPALITY</td>
<td>UNITED ARAB EMIRATES</td>
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<td>Mohamed Hasan AL SAEEDEI</td>
<td>HEAD OF DESIGN AND TECHNICAL REVIEW SECTION</td>
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<td>Dr. Ahmed Hassan Ali ABDELAWAD</td>
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<td>Dr. Ammar M. K. JARRAR</td>
<td>IRRIGATION EXPERT</td>
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</table>

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**KEYWORDS:**
Abu Dhabi 2030 Vision, Value Engineering, Value Engineering Guidelines, Benchmarking study

**ABSTRACT:**
Rapid urbanization around the world results in increased demand for the development of road and infrastructure facilities creating enormous opportunities for the construction industry. In line with Abu Dhabi 2030 vision, Abu Dhabi Government is also developing a world class road and infrastructure network. To deal with the increased demand, a number of initiatives are being taken by the industry experts. The Municipal Infrastructure and Assets Sector (MIAS) of Abu Dhabi City Municipality (ADM) recognized Value Engineering (VE) as an effective powerful tool for achieving an efficient, cost effective and sustainable road and infrastructure development.

The VE practice has long been applied in ADM and requirements were included in the Consultant Procedure Manual 2001 edition. As a strategic objective for Abu Dhabi government to implement VE principles in Abu Dhabi projects, to achieve more sustainable and value - maximized infrastructure projects, the ADM designated the development and update of existing VE requirements. As an integrated action, ADM established a team with internationally certified VE specialists to develop the updated Value Engineering Guidelines (VEG). In view to conduct the practical application for VE study for different roads and infrastructure projects, a systematic Benchmarking Study² has been conducted to consider the latest and best practices in VE applications and guidelines, as developed by many national and international authorities.

This paper presents ADM conventional VE approach, updated Value Engineering Guidelines¹- A Uniform and Structured Approach, key highlights of the guidelines and its development process. This paper will also discuss some challenges encountered during preparation of the guidelines as well as implementation challenges of the updated guidelines and the way forward for promoting VE on a wider spectrum.
Abu Dhabi City Municipality’s Initiatives for the Application of Value Engineering in Road and Infrastructure Projects


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1 INTRODUCTION

Rapid urban growth throughout the developing world is seriously outstripping the capacity of most cities. As a result, increased demand for the development of road and infrastructure facilities creates enormous opportunities for the construction industry; however the construction industry faces many challenges to meet this demand due to limited resources, constrained budgets, tight construction period, maintaining adequate quality and safety needs, etc. In such situations, the Client’s and Government’s priority is to judiciously spend their budgets and optimize returns on investment to meet the ultimate needs of their residents. To deal with the increased demand, a number of measures are being taken by industry experts around the world.

In line with Abu Dhabi 2030 vision, a roadmap for the Emirate’s economic progress and developing a world class road and infrastructure network, the Municipal Infrastructure and Assets Sector (MIAS) of Municipality of Abu Dhabi City (ADM) recognized Value Engineering (VE) as an effective and powerful tool for achieving an efficient, economical and sustainable road and infrastructure development. A number of initiatives related to VE have been taken by ADM such as preparing policies and procedures, conducting workshops and awareness sessions, etc. Taking this initiative further, a professional VE certified team of technical specialists of ADM has prepared a Value Engineering Guidelines (VEG) in view of implementing and promoting a uniform and structured approach to VE across the Abu Dhabi Road and Infrastructure projects.

2 ADM CONVENTIONAL APPROACH TO VE

The practice of VE has long been applied in ADM whereby requirements were briefly included in the 2001 edition of Consultant Procedure Manual (section 3.2.1.2). Focus was given on cost effectiveness and identifying reliable, potential savings for various project elements.

Later, in 2014, the VE requirements were further elaborated and included in updated version of the same manual (section 2.2.3). These guides were based on conventional approach where the VE process was embedded within the design development and review processes and applied during concept, preliminary and final design stages of the projects.
The main features of the ADM conventional VE approach are summarized below:

- As part of ongoing analysis of cost effectiveness, VE is considered in the design of all elements of a project. VE objectives were defined as to improve quality, minimize cost, reduce construction time, improve constructability, enhance safety and meet environmental goals.
- The VE multi-disciplined team of individuals is to undertake an independent review of the project to ensure that the above objectives are met.
- The VE review process needs to be undertaken as early as possible after the basic design elements and preliminary cost information have been developed – typically after the preferred alternative has been identified.
- The VE team is to carefully analyze the merits of each alternative considering the project as a whole.
- VE analysis recommendations shall be well documented throughout the study process.
- The consultant is to present the recommended VE alternatives to the ADM and other stakeholders, as required. The VE study is then to be submitted as a formal report to the management for approval and recommendations for implementation.
- Value management workshops for the project may also be required to be undertaken at different design stages. After the workshops, the consultant and design team are to jointly determine which of the recommended value management proposals should be implemented. These proposals are to be submitted as a formal report to the ADM for review and approval.

From 2011 to 2015, ADM VE implementation has resulted in achieving a total cost savings over AED 210 millions. Refer to Table 1 for details.

Table 1. Cost Reduction in Some of Roads & Infrastructure Projects

<table>
<thead>
<tr>
<th>Elements of Road &amp; Infrastructure Projects</th>
<th>No. of Projects</th>
<th>Savings (millions AED)</th>
<th>Savings (Percent %)</th>
<th>VE Solution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Street Lighting</td>
<td>4</td>
<td>8</td>
<td>52</td>
<td>Replacing Conventional lantern by LED lantern. Decreasing the power supply by decreasing the laying cables loops &amp; backfilling disposal of materials &amp; supply and install curved street light poles with bracket luminaires by decreasing the spaces between the poles</td>
</tr>
<tr>
<td>Road Pavement</td>
<td>10</td>
<td>115</td>
<td>14</td>
<td>Introducing Geo-grid in sustainable pavement design Extending Pavement Service Life &amp; Decreasing Pavement Thickness, Increasing Pavement Durability, decreasing construction and life cycle cost, construction time, carbon emissions.</td>
</tr>
<tr>
<td>Landscape</td>
<td>3</td>
<td>45</td>
<td>32</td>
<td>Study resulted in removing certain elements in the design due to compliance with local authorities (UPC) requirements. Reduced Palm tree sizes as well as thickness of granite. Granite and Natural Stone pavers was replaced with stamped sand. The cost study has also impacted various landscaping elements such as landscaping materials, irrigations network, shading, planting, furniture, lighting, play equipments, etc.</td>
</tr>
<tr>
<td>Stormwater</td>
<td>14</td>
<td>38</td>
<td>32</td>
<td>Reduced peak runoff helps reducing network size. Used surcharged network approach and hydraulic modeling to optimize network. Reducing dependence on the pumping/lifting facilities as well as combining drainage system with the groundwater control measures where possible. Also system layout was optimized.</td>
</tr>
<tr>
<td>Structures</td>
<td>2</td>
<td>5</td>
<td>15</td>
<td>Structural system of bridges was modified and design elements were optimized. Cost saving realized as a result of reduction of concrete quantities.</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>33</strong></td>
<td><strong>211</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Compiled from Value Engineering Benchmarking Report (2011-15)
3 DEVELOPMENT PROCESS OF THE VALUE ENGINEERING GUIDELINES

Taking a step further and as a strategic objective for Abu Dhabi government to implement VE principles in Abu Dhabi projects, to achieve more sustainable and value-maximized infrastructure projects, the ADM designated the development and update of existing VE requirements. As an integrated action, ADM established a team with internationally certified VE specialists to develop the updated Value Engineering Guidelines (VEG).

During development of this guide, information was collected through a comprehensive literature review including documents from various transportation agencies in the USA, Canada, UK, Australia and other local authorities. Also, insight gained from the knowledge and personal experiences of the VE team assigned for preparing these guidelines and feedback from ADM internal stakeholders, was considered, where appropriate.

The development and implementation process of the VE Guidelines included the following steps:

- Forming Team
- Preparing action plan
- Obtaining training on VE from authorized and relevant sources
- ConductingBenchmarking Studies
- Analyzing findings of Benchmarking Studies and formulating recommendations
- Internal and external review and incorporating feedback and suggestions received
- Completing final draft of the guidelines
- Conducting a pilot study to validate the guidelines
- Planning to conduct technical and training workshops
- Planning to issue final guidelines for implementation

4 KEY HIGHLIGHTS OF THE GUIDELINES

The VE Guideline is divided into 6 chapters. Chapter 1 describes the purpose and scope, chapter 2 includes historical background, concepts, objectives and benefits of VE and chapter 3 briefly defines phases and design stages of a typical ADM project.

Chapter 4 and 5 are the core of the guidelines. Application criteria of VE during different phases of a project are described in chapter 4 while chapter 5 focuses mainly on uniform and structured procedures for conducting a typical VE study. Chapter 5 also specifies the ADM’s requirements related to VE team composition, team qualifications, role and responsibilities of the team, etc. In addition, the objectives, purposes, major activities, and typical outcomes of all six phases of a typical VE workshop are included in this chapter.

Chapter 6 is intended for creating awareness about VE education and professional certification programs. Also, a list of references, basic VE definitions, checklists & forms, supplementary information on workshop stages, etc. are included in the appendices of the guidelines.

The key highlights of the guidelines are given below:

PURPOSE

The purpose of VE guidelines is to update the existing VE practices, develop and establish policies as well as procedures for ADM to select, study, report and implement VE studies related to infrastructure projects. The guidelines also aim to promote VE culture through consistent, uniform, and structured VE approach adopted during the various applicable phases of projects to ensure:

- VE program and policies are established, updated and followed
- Coordination with other ADM divisions to ensure the VE process is integrated in the planning, design, construction and maintenance program processes, where warranted
- Ensure that the VE analyses are planned and conducted in accordance with the policies and procedures set in these VE guidelines, and that recommendations developed and implemented for each project are properly documented.
- Monitoring, evaluation, and reporting to ADM management the results of the VE analyses that are conducted and the recommendations implemented for each project.
CONCEPT OF VE

The VE guidelines presented are conceptually derived from SAVE International\(^5\) approach of Value Management (VM). However, where necessary, the approach has been customized to reflect the ADM requirements and project frameworks.

In the context of an ADM infrastructure project and processes, VE study is an organized and structured study of the function of all and/or specific components that make up the project. A VE study covers a broader area than just a life-cycle cost analysis. The VE study process, defines a sequence of activities that are undertaken during a VE study; before, during, and following a workshop. During the VE workshop, the VE team learns about the background issues, defines and classifies the project functions, identifies creative approaches to provide the functions, and then evaluates, develops, and presents the VE proposals to key decision makers. It is the focus on the functions that the project must perform that sets VE apart from other quality-improvement or cost-reduction approaches.

VE helps achieve an optimum balance between function, performance, quality, safety, and cost. The proper balance results in the maximum value for the project. Value is the reliable performance of functions to meet customer needs at the lowest overall cost. Value is proportional to the ratio of function over cost as indicated below:

\[
\text{Value} \propto \frac{\text{Function}}{\text{Cost}}
\]

Function is what the project is expected to do; Cost is the expenditure needed to create it.

In other words, VE is the systematic application of recognized techniques by multidisciplinary team(s) that identifies the function of a project, establishes a worth for that function, generates alternatives through the use of creative thinking, and provides the needed functions, reliably, at the lowest overall cost.

BENEFITS OF VE

ADM recognizes the need for prudent use of resources and revenue while providing quality roads and infrastructure projects. VE is a function-oriented technique that has proven to be an effective management tool for achieving improved design, enhanced construction, and cost-effectiveness in various infrastructure elements. Implementation of VE optimization concepts is a major objective in ADM. It is more than just employing the right designs and construction techniques. Considering Performance, Life Cycle Cost, Quality and Safety, Energy and raw materials consumption, sustainability, stakeholder’s engagement, as well as minimized impacts on natural environment and public health are major considerations. To achieve these targets ADM has included the implementation of VE optimization as one of the major objectives to be fulfilled in roads and infrastructure projects.

Some of the key benefits realized are given below:

- Improvement of project value, performance and quality
- Reducing the time to complete the project
- Providing the needed functions safely, reliably, efficiently, and at the lowest overall cost
- Eliminating unnecessary design elements
- Fostering innovation and improving productivity
- Promoting sustainability
- Managing stakeholders expectations

CRITERIA OF VE APPLICATIONS IN PROJECTS

Every project has several phases of development known as life cycle phases. These phases are varied depending upon the type of projects but all follow the same basic steps. Typically ADM procure projects through two types of contracts i.e. Design-Bid–Build and Design-Build. Application of VE study can be undertaken through the various phases of a project life cycle. The main phases of the ADM design-bid–build project life cycle are Planning and Study (or Pre-Concept) Phase, Design Development Phase, Construction Phase and Operation and Maintenance Phase. The design development phase includes Conceptual Design Stage, as applicable, Preliminary Design Stage and Final Design Stage.

VE is applicable at any phase or stage in a project’s life cycle, but greater potential savings can be achieved the earlier the VE Study is performed. VE should, therefore, be applied as early as possible in the project life cycle. Early VE tends to produce greater savings (or cost avoidance) because that is where most of the costs are committed to—
During project planning, VE can make a major impact on the life-cycle cost (LCC) of a project. Adjustments to the program at this point have very little, if any, disruptive impact on schedule and redesign costs. Consequently, the project should proceed with fewer changes and with a greater understanding by all parties of what the final function and space allocations will be.

Most effective application of a VE study is realized early in the design phase. Changes or redirection in the design can be accommodated without extensive redesign at this point, thereby saving the owner/user/stakeholder's time and money.

At the concept design stage, performing a VE study, to develop design alternatives, defines the project’s functions and achieves consensus on the project’s direction and approach. By conducting a VE study exercise at this stage of the design, the project needs are identified in the common language of functions and miscommunication as well as redesign efforts are minimized.

At the preliminary and final design stages, performing a VE study analyzes the potential alternatives of the selected option, drawings and specifications as well as concentrates on economics and technical feasibility. It also considers selection of equipment and materials, methods of construction, phasing of construction, and procurement. The goals at this stage of design are to minimize construction costs and reduce the potential for construction claims, satisfy stakeholder needs, and review the design, equipment, and materials used.

In general, selecting projects requiring VE study and its time frame is based on project estimated cost, project size and/or complexity. These include projects that have critical constraints, difficult technical issues, expensive solutions, external influences, and complicated functional requirements, regardless of the estimated project cost.

For ADM, VE analysis may be initiated during the design stage on any project or process when it is felt that there are sufficient potential cost savings or added value to justify the cost of VE analysis. Various international authorities have specified different threshold criteria based on project value for undertaking VE studies; however, as per current practice in ADM, Table 2 addresses the criteria that shall be used to determine which projects require the performance of a VE analysis.

### Table 2. VE Implementation Thresholds

<table>
<thead>
<tr>
<th>Project Category</th>
<th>Project Estimated Cost (AED millions)</th>
<th>Design Stage of VE Study Selection</th>
<th>Design Team of Project Consultant</th>
<th>Independent Design Team of Project or other Consultant</th>
<th>Independent VE Certified Consultant</th>
<th>Required Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>≤ 10</td>
<td>Concept</td>
<td>-</td>
<td>-</td>
<td>General VE Principles</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>10 &lt; Cost &lt; 120</td>
<td>Concept</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Formal VE Study Process</td>
</tr>
<tr>
<td>3</td>
<td>≥ 120</td>
<td>Concept</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Formal VE Study Process</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Preliminary</td>
<td>-</td>
<td>-</td>
<td>-</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>Detailed</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

Compiled from VEG\(^1\) (2017)

In some projects, subject to ADM management approval, application of above criteria may vary based on the potential for cost savings in comparison to the cost of the VE analysis or the potential to improve the projects’ performance or quality.

During project construction phase, VE improvements are still possible using Value Engineering Change Proposals (VECP). The objective of the VECP is to encourage contractors to investigate improved construction methods and materials through VECPs. The change is intended to reduce cost (initial and/or life-cycle) but still meet or exceed all necessary functions including performance, safety, aesthetics, operations, quality and time.
During project Operation & Maintenance Phase, applicability of VE study depends on the type of contracts and assessed on a case by case basis. In cases where a VE study is applicable, it may not be necessary to follow a formal VE study process (Figure 3); rather, general VE principles or conventional VE concepts can be adopted. The VE Study may include considering cost benefit analysis of project elements, application of new technology, alternative materials, equipments, repair methodology, etc.

Design-build is a project delivery method in which either design and construction are provided from a single source or the concept design is developed by a different source prior to tendering the design-build project. Since VE study optimizes design and promotes project value, construction through the design-build project delivery method is compatible with the goals of VE. Furthermore, VE will augment the benefits of design-build since its focus extends beyond cost-cutting and creativity in construction. For Design-Build projects, VE criteria and thresholds specified in Table 2 are applicable. Implementing VE within the design-build process provides the following benefits:
- Generates cost shared savings for both the owner and the design-builder.
- Spur innovation in construction, since the terms of the owner’s RFP may be re-evaluated or modified.
- Minimizes risk to both the owner and the design-builder due to the consensus approach to the project.
- Increases likelihood that the resulting project will satisfy owner and user needs.

VALUE STUDY PROCESS

Value Study Process is a systematic and structured action plan for conducting, documenting of results and implementation of the VE Study. The VE Study includes the following three major stages:
- Pre – Workshop Stage
- Workshop Stage or Value Job Plan (The Six – Phase Work Plan)
- Post Workshop Stage

The Value Study process flow chart is presented in Figure 3 below:

![Figure 3. Value Study Process Flow Diagram](image)

The purpose of Pre–Workshop Stage is to refine the problem and prepare for the value study. The fundamental question that needs to be answered during the pre-workshop stage is “what should be done to prepare for the Value Study?” the main tasks of this stage are formation of VE team, coordination with stakeholders, data collection, and information modeling. The post workshop stage is to monitor the approval process and implementation of the action plan. Post workshop stage is divided in to implementation phase and follow up phase.

The Workshop stage or Job Plan is the major part of the Value Study, which includes the following sequential six phases:

I. Information Phase: The team reviews and defines the current conditions of the project and identifies the goals of the study.

II. Function Analysis Phase: The team defines the project functions using a two-word active verb/ measurable noun context. The team reviews and analyzes these functions to determine which need improvement, elimination, or creation to meet the project’s goals.

III. Creative Phase: The team employs creative techniques to identify other ways to perform the project’s function(s).

IV. Evaluation Phase: The team follows a structured evaluation process to select those ideas that offer the potential for value improvement while delivering the project’s function(s) and considering performance requirements and resource limits.

V. Development Phase: The team develops the selected ideas into alternatives (or proposals) with a sufficient level of documentation to allow decision makers to determine if the alternative should be implemented.

VI. Presentation Phase: The team leader develops a report and/or presentation that documents and conveys the adequacy of the alternative(s) developed by the team and the associated value improvement opportunity.

For each of the above phases, the objective, purpose, common activities and expected outcome are illustrated in the VE guidelines.

5 CHALLENGES

Since locally VE is a new industry there is significant lack of awareness about formal VE approach. In addition, due to lack of trained professionals there is significant shortage in qualified resources available for conducting VE Studies. There is also a wrong perception among the local engineering/construction industry that VE is just a cost cutting exercise and which may unnecessarily result in project delays and create project variations and other contractual issues. Some other challenges include the need for increased coordination among stakeholders, lack of commitment towards adopting a formal VE process as well as a general resistance to change from the ongoing practice.

With respect to the above stated issues, some challenges were also faced in preparation of the guidelines. These include lack of locally published literature, difficulties in coordination with other stakeholder, difficulties in obtaining peer review from local industry practitioners and other relevant international professional organizations etc.

7 THE WAY FORWARD

Abu Dhabi Municipality has been practicing VE as a mandatory requirement for all projects however the application process is embedded within the design stages and usually no separate formal VE workshops is conducted. The final draft of VE guidelines have been developed which was internally reviewed by ADM and many international engineering consultants. The final draft was also reviewed by SAVE International. Valuable feedback and suggestions received were noted and incorporated in the guidelines, where warranted.

As part of strategic plan, ADM is currently in the process of conducting a pilot study on an Infrastructure project to validate the guidelines for its practical applications. The findings of the pilot study would be analyzed and, if required, incorporated in the guidelines. For effective implementation of the VE guidelines, a number of technical and training workshops would also be conducted prior to circulating the guidelines to the industry experts, consultants, contractors, developers and other stakeholders.
In order to promote VE and to encourage wider implementation of VE, the guidelines will be forwarded to Abu Dhabi Quality and Conformity Council (QCC) for formal publication and adoption by all relevant governmental entities across the Emirate of Abu Dhabi.

8 CONCLUSIONS

Almost all entities are vigilant and looking forward for opportunities to reduce the cost of their projects while still maintaining the quality. In efforts to lowering the cost of the projects, a conventional approach to VE is generally adopted by various public sector and private sector entities. ADM have been implementing VE, as one approach, to help cost-effectively deliver needed infrastructure and satisfy customers. VE was initially applied in view of reducing overall construction costs. During the process of applying VE, a number of VE opportunities are often missed or overlooked due to a lack of adopting a proper, uniform and structured VE approach. As a result maximum benefits of VE concepts are not fully utilized in projects. However, it is recognized that greater benefits can be realized if a systematic VE process is adopted in the development of the project. Abu Dhabi Municipality has reviewed their previous VE practices and prepared the updated VE Guidelines in view to have a consistent, uniform and structured approach for implementation in Infrastructure projects across the Emirate of Abu Dhabi. The structured VE approach adopted from Save International is based on the concept of functional analysis which will help to achieve maximum benefits in Infrastructure projects. At the same time this will promote a uniform and structure approach to VE on a wider spectrum.

ACKNOWLEDGEMENTS

We would like to express our gratitude to Abu Dhabi Municipality (ADM) who involved us in preparation of Value Engineering Guidelines as well as provided this wonderful opportunity to present this paper on behalf of ADM. Special thanks to Eng. Majed Abed Alkatheeri, Eng. Husain Mohsen Alsaeedi, Eng. Eiman Saeed Alameri and the entire ADM VE team for their hard work and dedication in developing the VE Guidelines.

Besides, we would like to express our sincere thanks and appreciation to all ADM reviewers, SAVE International, Parsons International, and Hyder Consulting Engineers for their valuable suggestions and provided feedback during the review process of the VE guidelines.

REFERENCES

3. Abu Dhabi City Municipality (Nov. 2014). Consultant Procedure Manual for Design Consultancy Services, ver. 2.0, sec. 2.2.3, page no. 8 to 1, Department of Municipal Affairs, Internal Roads and Infrastructure Directorate, Abu Dhabi.
5. SAVE International (Mar. 2015). Value Methodology Standard, USA.
### ABSTRACT:

Value Engineering is an intensive, interdisciplinary problem solving activity that focuses on improving the value of the functions that are required to accomplish the goal, or objective of any product, process, service, or organization. It can be applied to any process and project with the main objective to optimize resources – material, manpower and time. Conventional construction approach towards executing a project can lead to a negative impact on the resource management. The impact increases with the increase in complexity of the project. The project under discussion is “Rehabilitation, Strengthening and Four Laning of Jammu to Udhampur Section from km 15.00 (on Jammu Bypass) to km 67.00 of NH 1-A in the State of Jammu & Kashmir. The complexity of the projects lies in difficult hilly terrain, unpredictable weather conditions, difficult access and unavailability of material and manpower. There were 78 bridges under 8 different types in total to be constructed on the project and each had different span arrangements making the entire construction process extremely complex and resource consuming. A total of 3 kms of twin tunnel was to be constructed in a weathered rock class condition requiring a combination of plain shotcrete and tunnel lining as per initial design. In order to fast track the project, value engineering efforts were directed towards standardizing bridge spans and using steel fibre reinforced shotcrete to eliminate tunnel lining. In this paper we have discussed the value engineering efforts and the effective implementation of it through a case study.
ACHIEVING SUCCESS THROUGH VALUE ENGINEERING IN ROAD PROJECTS

– A CASE STUDY

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1 INTRODUCTION

Value Engineering (VE) is a management technique that seeks the best functional balance between cost, reliability and performance of a product, project, process or service. Value engineering is a powerful problem-solving tool that can reduce costs while maintaining or improving performance and quality requirements. Value engineering can improve decision-making that leads to optimal expenditure of owner funds while meeting required function and quality level. The success of the VE process is due to its ability to identify opportunities to remove unnecessary costs while assuring quality, reliability, performance, and other critical factors that meet or exceed customer’s expectation.

The project being discussed in this paper as a case study for successful completion of a highly complex road project using various techniques of value engineering is: “Rehabilitation, Strengthening and Four Laning of Jammu to Udhampur Section from km 15.00 (on Jammu Bypass) to km 67.00 of NH 1-A, Jammu & Kashmir, Total Lane 256.64.58 kms”.

Various areas where value engineering is demonstrated in the project are listed as below:

1. Standardization of bridge spans
2. Combination of RCC & RE wall
3. Use of steel fibre reinforced shotcrete (SFRS)
4. Need of sand washing plant

2 STANDARDIZATION OF BRIDGE SPANS

Jammu Udhampur Road Project had a total of 78 bridges – having 8 different types of superstructures and varying span lengths as per the Concessionaire Agreement (CA). It would have been a major challenge to execute such varying types of superstructures and spans. The amount of resources required in design, formwork and shuttering of these bridges would have been enormous both in terms of time and money. The availability and mobilization of resources in J&K State is a challenge. The obvious plan to mitigate the issue was to look at options of reducing the types of superstructure and standardization of span lengths.

As per the CA proposal the super structure was to be constructed using eight different types: Solid slab, Multiple Slab, Solid Slab with T beam, Box girder, PSC box girder, PSC box girder with T Beam, Slab with T Girder and Steel Structure. The spans were varying for each bridge as per the topography. It was discussed and agreed that if span arrangement could be standardized for majority of structures then we will be able to reduce structure to structure dependability and save substantial time by converting a sequential activity into parallel activity. There was a provision in CA that contractor could adopt alternate design for structures keeping the clear water way same. The objective of alternate design was to save time and do value addition in terms of economic feasibility in the project.

The design and survey teams relooked at all the span arrangements to come up with three combinations as standard spans:

1. 20 m spans to be done with precast I girders (Figure1)
2. 40 m spans to be done with composite steel girders (Figure 2)
3. 70 m spans to be done with balanced cantilever using bridge builder (Figure 3)

In totality, there were 340 spans which were to be constructed in the project. There was a plan to take up 20 spans at a time. Considering the average cycle time for construction of super structure below the slab from staging till concreting and curing as 45 days. There is a total saving of 765 days in the construction of structures.

Total no spans = 340 Nos
Spans taken up at one time = 20 Nos
Time taken for one span = 45 days
Time saved is = (340/20) x 45 days = 765 days
The above saving was achievable because of value engineering done at various levels and converting the activities to be independent of each other.

Figure 1. I-girders for 20 m span bridges

Figure 2. Steel girders for 40 m span bridges

Figure 3. Balanced cantilever construction with bridge builder for 70 m span bridges
3 COMBINATION OF RCC & RE WALL

The project being in hilly terrain required construction of more than 11 km of earth retaining structures. The retaining wall height varied from 2 m to 22 m. There was a constraint to the Right of Way due to the hilly terrain and getting ledge distance on hill slopes was a real challenge. The construction of RCC retaining wall would have been very uneconomical and time consuming for heights more than 6 m. The width of the foundation for a 22 m high retaining wall would be around 18 m. The foundation width could not be extended toward the hill side for excavation as it would destabilize the hill slope and the movement of traffic in the existing road. Constructing counterfort type retaining wall (Figure 4) could have been an alternate option. But that would have only reduced the thickness of the wall and not the width of the foundation. The backfilling of counterfort wall is also a challenge.

As an alternative, a structure with a combination of RE Wall constructed over RCC retaining wall was envisaged for 22 meters’ height retaining wall (Figure 5). A proposal for constructing 10 m high RE Wall over 12 m height of RCC retaining wall was discussed with the structural design engineer. The RCC retaining wall was designed to carry load of extra surcharge of the RE Wall. L shaped RCC retaining wall below RE wall was designed for certain locations where there was constraint of getting ledge space as well as closing the movement of the existing traffic.

There was a considerable saving in time and cost observed with this design change adopted in the project. Approximately 42% saving in cost was observed when the two systems were compared for a 20 m height retaining wall.

Cost of conventional RCC Retaining wall of 20 m height – Rs. 4.54 lacs per running metre
Cost of combined RCC Retaining wall & RE Wall of 20 m height – Rs. 2.61 lacs per running metre

Figure 4. Counter fort Retaining Wall

Figure 5. RE Wall + Retaining Wall Combination
4. USE OF STEEL FIBRE REINFORCED SHOTCRETE

Jammu Udhampur Project had about 3 kms of tunnel. The initial design of tunnel support was a combination of wire mesh embedded in Plain shotcrete further supported by final tunnel lining. This process is a time-consuming process as it needs the wire mesh to be physically put in place and then plain shotcrete to be sprayed. As an alternative, it was decided to design the tunnel support with steel fibre reinforced shotcrete (SFRS) thus expediting the works as well as eliminating the requirement of tunnel lining.

In the design of tunnel lining, the fundamental principle is mobilization of the strength of rock mass - it relies on the inherent strength of the surrounding rock mass being conserved as the main component of tunnel support. The main idea is to use the geological stress of the surrounding rock mass to stabilize the tunnel. Primary support (shotcrete) is directed to enable the rock to support itself. This is the core of the NATM – The New Austrian Tunneling method (NATM) and Norwegian Tunneling Methods used in tunnel excavation.

The large deformations that occur in the ground can easily overload brittle material such as plain Shotcrete / Reinforced Concrete and lead to failure. The combination of fibers and Shotcrete (SFRS) makes a composite material of sufficient ductility to accommodate large ground deformations several hours after the application. These deformations are accommodated from their very onset, when they develop most rapidly, without damage to the SFRS, so that the lining does not lose any of its initial capacity. The subsequent gain in strength of the lining with time is enhanced by the lessening of ground deformation. From this point of view, SFRS is an ideal material for tunnel lining.

Considering the strata as sand stone where the Rock Mass Quality “Q”, Excavation Support ratio of 1.0 and tunnel width of 10.5 m, the choice of SFRS mix with Energy Absorption value of 700 Joules is preferred. The proposed SFRS thickness of 50mm for the initial layer was based on immediate support capacity requirement of tunnel Lining. The Technical Brochure of Dramix gives the requirement of steel fibers as 25kg for SFRS mix with Energy Absorption value of 700 Joules. When 35 kg dosage is used the energy absorption value will change from 700 to 1000 Joules. Higher the energy absorption values better will be the performance of SFRS mix especially in areas of high seismic zone like Jammu & Kashmir.

The fibers reinforce the shotcrete homogeneously and give a resistance to tensile stresses at any point in the shotcrete layer. SFRS gives increased load bearing capacity due to the redistribution of stresses. The removal of mesh leads to an increased shotcrete bond to the substrate. SFRS is an excellent corrosion resistant material and the network of fibers in the shotcrete eliminates the problem of spalling due to rebar corrosion. SFRS has excellent control of cracks giving less leakage. SFRS avoids “shadowing” or voids behind mesh. SFRS produces higher quality in-place shotcrete whereas shooting through wire mesh can be very difficult, resulting in honeycombing, poor consolidation, shadowing, embedded rebound and poor bond to the mesh. Same is true for reinforced concrete lining that require special gantry for casting and highly skilled personnel to carry out its construction.

The use of SFRS reduces the time cycle for tunnel construction. Use of NATM method gives enough warning on face advance and remedial measures to be adopted, the thickness of SFRS can easily be varied as per need. This is not easily possible with Reinforced Concrete Lining due to the rigid profile and gantry being used for its construction. The safety of underground construction workers is of utmost importance. SFRS provides instant support. There is no need to install mesh in unsupported ground which is a time consuming and risky operation. A completed view of tunnel is shown in Figure 6.

The overall cost optimization and time saving because of change in design of tunnel support is as given in Table 1 below:

| Approx. time required for 5 m tunnel completion with conventional tunnel support | 3 days | Total saving in time for 3 kms = 600 days |
| Approx. time required for 5 m tunnel completion with SFRS system | 2 days |
| Approx. cost for 5 m tunnel completion with conventional tunnel support | 15 lacs | Total saving in cost = Rs. 18 cr. |
| Approx. time required for 5 m tunnel completion with SFRS system | 12 lacs |
5. NEED OF SAND WASHING PLANT

The fine aggregates requirement in the project was around 2.5 lakh cum for the concreting work. The quality of natural sand available in the region was not as per the standards to be used for concreting works in the project. Since the crusher plant was operational for production of aggregates the availability of crushed sand was in abundance. The source of aggregates, being river bed material the crushed or crushed sand quality is also weather dependent. It was observed that even the crushed sand had a very high clay content, which prevented it from being directly utilized for concrete production. The chances of use of natural or crushed sand without any treatment would have led to increase in cement content per cum in the concrete mix design due to presence of silt and clay in the sand. To make the crushed sand suitable for the concrete works, provision for washing of sand was required to remove silt and clay. Hence the proposal to install a sand washing plant was initiated in the project (Figure 7 & 8).

The initial silt content in crushed sand was approx. 8% which was reduced to approx. 2% after washing. After washing, the crushed sand was tested for grading and observed that it falls in Zone II which is very good for concrete design mixes. Hence the concrete mixes were redesigned using washed crushed sand. This gave better results compared to concrete mixes with natural sand. This helped us to maintain the quality standards required by the project and have a better control on the fine aggregate throughout the project. By installing the washing unit in the project a saving of at least average 1.02 kg/cum of concrete for each grade of concrete was observed in the project. Considering the rate of cement at 5.5 Rs/Kg the net saving on account of concrete was approximately 3.33 crores.

<table>
<thead>
<tr>
<th>Sr No</th>
<th>Grade of concrete</th>
<th>Cement content in concrete mixes with natural sand in kg/cum</th>
<th>Cement content in concrete mixes with washed crushed sand in kg/cum</th>
<th>Difference</th>
<th>Unit Rate of cement per Kg</th>
<th>Concrete Qty in the project in cum</th>
<th>Saving in cost of concrete in lakhs</th>
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</thead>
<tbody>
<tr>
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</tr>
</tbody>
</table>

Average cement saving in Kg per cum 12

| Table 4(2): Saving in concrete cost by using washed sand |
9. CONCLUSION

Value engineering is a methodology used to analyze the function of the goods and services and to obtain the required functions of the user at the lowest total cost without reducing the necessary quality of performance. The essential difference between conventional cost cutting and value engineering is that it involves reducing the cost by improving the functionality through lesser consumption of energy in terms of manpower, materials and machines. The decision to install sand washing unit in the project has helped to improve the quality of concrete as well as an impact on the saving in cost of concrete to the extent of 3 crores. Similarly, the use of steel fiber reinforced shotcrete in the tunnels for eliminating the conventional tunnel lining has saved the time of construction in the project by approximately 40%. The decision to construct the combination of RCC retaining wall and reinforced earth retaining wall has save the time by maintaining the existing movement of traffic and a cost saving of approximately 42%. Thus one can say use of effective value engineering helps the project for smooth completion with good margins and branding.

10. ACKNOWLEDGEMENT

The author wishes to express his sincere thanks and profound sense of gratitude to all who are actively involved in completing the project. The contents of this report reflect the views of the author, who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the NHAI or State PWD and Indian Road Congress. This report does not constitute a standard, specification, or regulation.
Data-Driven Benefit Analysis of a Major Bridge Expansion Project in Tehran

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ABSTRACT:
Similar to all other major metropolitan regions, traffic congestion is one of the most problems in Tehran. In addition to millions of hours of wasted citizen’s times every year, traffic congestion is the main cause of air pollution and increased fuel consumption. Studies show that when traffic speeds fall below 20 km/h, pollution emission rates and fuel use can increase by a factor of ten which negatively impacts air quality in urban road network. Identifying and fixing bottlenecks is one approach towards congestion mitigation. This paper analyzes the effect of expanding Sattarkhan bridge which is one of the most problematic traffic bottlenecks in Tehran. Travel time and speed data was collected using traffic sensors deployed in the study corridor both before and after the bridge expansion. This data was blended with the outputs from pollution emission and fuel consumption models calibrated for the passenger vehicles in Tehran, in order to calculate savings in travel times and reduction in pollutants and fuels as well as economic benefits of the project. In summary, the additional capacity provided by the bridge expansion translates into 4700 hours of passenger time saving per day. The results show reduction in emission of Carbon Dioxide (CO2) by 18%, Carbon Monoxide (CO) by 16%, Unburned Hydrocarbons (UHCs) by 19%, Nitrogen Oxides (NOx) by 12% and petroleum usage by 17%. However Particulate Matter (PM) levels had 6% increase. Average economic benefit of the project was then assessed by converting all benefits to monetary values.

KEYWORDS:
Bluetooth Traffic Sensor, Impact Analysis, Benefit Study

1. Introduction

Most of the large and modern cities around the world, take advantage of new sensing and monitoring technologies to enhance urban management solutions including mobility and air pollution. These solutions work best if they are integrated and coordinated by a central entity which usually is the municipality. Traffic data is a vital element of any smart and successful transportation management system and is used for mobility management, before-after studies, and producing new solutions for traffic bottleneck and congestion related problems. Traffic and air pollution are the two main problems of major metropolitans and are closely related to each other. Vehicular traffic is a major cause for wasting millions of hours of the citizens’ times and also for increasing air pollution and fuel consumption. These impacts translate into burden and cost to both the human society and the economy.
The detrimental effects of air pollution on Tehran citizens cannot be ignored. Because of this, controlling the pollution caused by cars (as the main source of urban air pollution) has always been the subject of concern. Great efforts have been made to implement prevention strategies to mitigate the problem in two general categories:

1. Supply side strategies including changes to the fleet and the network. Examples include road widening projects, implementing Compulsory Euro 2 emission standard, promoting electric motorcycles, and Hybrid taxis.

2. Demand side strategies including actions to reduce the number of cars in the streets. Examples include odd and even day traffic restriction project, expanding the traffic restricted zone, and petrol rationing.

The second category, because of its vast application, covers almost all vehicle classes in Tehran (passenger vehicles, pickups, and heavy trucks) and has a more significant effect compared to supply side strategies [1]. Traffic reduction is in the heart of transportation systems management decisions in Tehran. The pollution emission rate (and fuel consumption) in speeds below 20k/h, is ten times more than the speeds above 40k/h. Thus, eradicating the traffic leads to less emissions of pollution and at the same time decreases fuel consumption. This has many economic benefits and also improves air quality in borderline residential areas [4-2]. In order to measure the impact of any new policy or project on traffic and air quality, Tehran municipality has started using Bluetooth traffic minorig detectors developed by Mana-Rayka company. This paper shows the results of a bridge expansion project in Tehran, using data collected by the aforementioned sensors.

SattarKhan Bridge has been one of the major problematic bottlenecks in Tehran’s urban network for many years. The low functioning bottleneck not only is to blame for increased travel time and fuel consumption, but also is a hotspot for intense pollution generated by cars resulting in poor quality of the air around the bridge. The deterioration of the traffic in this area in the morning rush hours reached to the point that sometimes in the west to east direction of the Sheikh Fazlolla highway, queue lengths reach up to 3 kilometers and even more, and have cascading effect on other links like the Shahid Sodagar Bridge. Afternoon traffic in this bridge is also so congested that drivers traveling from west to the east parts of the Sheikh Fazlolla highway, and more specifically from Nasr neighborhoods to SattarKhan area get very frustrated and anxious. Adding lanes to both traffic directions on this bridge became inevitable and bridge widening project was started. The original width for each direction was 8 meters, and the bridge spans over 1200 meter distance. After widening, width for each direction is increased to 11 meters allowing to add one travel lane per each direction as shown in figure 1 [5 and 6].

2. Vehicle movement analysis

Bluetooth traffic sensors were deployed on both sides of the bridge before the widening project started, in order to capture hourly and daily traffic patterns. Approximately 1400 meters of the highway was covered by the sensors reporting data in real-time. Speed of vehicles during various hours in the day was analyzed. The study period had overlap with the Iranian New Year holiday so data was adjusted to account for the seasonal impacts of traffic. Data gathered from 13rd of January to 5th of March 2015 were regarded as data before
widening project and the data gathered from 3<sup>rd</sup> of April to 24<sup>th</sup> of April 2015 were regarded as data after the project. The amount of time devoted in various speed intervals before and after the project is shown in figure 2. Histogram of the speeds before and after the project implementation were created. Figure 2 shows comparative speed histograms for Saturdays both before and after the project.

![Histogram of speeds before and after the project](image1)

Figure 2. A comparison of the rate of the time consumed in each speed interval, for Saturdays before and after the widening. From south to north (right) and from north to south (left)

After completion of widening project, the time attributed to speeds above 40k/h has doubled [1]. Data was also analyzed to compare the distance travelled for each speed category, since pollution emission and fuel consumption correlate with the travel distance. Figure 3, shows the percent of distance in each speed interval.

![Histogram of distance by speed before and after the project](image2)

Figure 3. A comparison of the distance traveled in each speed interval, for Saturdays before and after the widening. From south to north (right) and from north to south (left)

After implementing the widening project of the bridge, it can be seen that majority of the distance traveled falls in the speed range above 40 k/h, while before the widening approximately 70% of the distance traveled is associated with speeds above 40 k/h. In addition, the average speed after widening for the route from south to north and vice versa have increased by factors of 1.6 and 1.3 times respectively. So as expected, the project potentially has significant effects on travel time, fuel consumption and pollution emission [1].

3. Time savings assessment

Reduction in travel time impact after the widening of the bridge is shown in figure 4. By multiplying the difference of travel time before and after the implementation of the widening project, in daily traffic, time savings is calculated. More details of the daily traffic ratio from south to north is provided in table 1.
Due to absence of reliable and accurate sources for the monetary value of user time in Iran, the value of time is assumed to be a factor of minimum hourly wage. According to the rules, the minimum wage in month for 164 working hours is 7,120,000 IRR\(^4\), so the hourly minimum wage is 43,410 IRR. Considering overall job portfolio and wage levels, weighted average hourly wage is considered 100,000 IRR in this study. Consequently, with all these information, the amount of daily savings caused by reducing travel time is calculated to be 470,341,000 IRR.

4. Pollution emission reduction and fuel consumption savings

To calculate the amount of pollution emission and fuel consumption before and after the widening of the bridge, numerical analysis and emissions modeling was conducted. A collection of 80 cars and taxis were examined by using a Portable Emissions Measurement System (PEMS) in real driving conditions. In every examination the mass amounts of gas pollutants output, engine operating parameters, and vehicle speed were measured in various traffic settings (with the air conditioning on/off). As a result, pollution emissions profiles were created and the ratio of emission factor (grams per km unit) and fuel consumption (liters per 100 kilometers unit) were calculated in various speed intervals. Results are summarized in table 2. Emission function and fuel consumption according to average speed have also been determined. Two samples of these functions are illustrated in figure 5. It is clear that emission and fuel consumption ratios at speeds below 20 k/h undergo significant reductions [7 and 8].

---

\(^4\) 1 USD = 38,000 IRR (Iranian Rials)
Table 2. Pollution emission and fuel consumption according to average speed for gasoline cars in routes without slope [3]

<table>
<thead>
<tr>
<th>Speed intervals(k/h)</th>
<th>0-5</th>
<th>5-15</th>
<th>15-25</th>
<th>25-35</th>
<th>35-45</th>
<th>45-55</th>
<th>55-65</th>
<th>65-75</th>
<th>75-85</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon dioxide</td>
<td>1190</td>
<td>335</td>
<td>238</td>
<td>182</td>
<td>150</td>
<td>139</td>
<td>133</td>
<td>143</td>
<td>127</td>
</tr>
<tr>
<td>Carbon monoxide</td>
<td>39.32</td>
<td>7.57</td>
<td>5.25</td>
<td>4.65</td>
<td>6.01</td>
<td>2.49</td>
<td>7.11</td>
<td>2.36</td>
<td>3.67</td>
</tr>
<tr>
<td>UHC</td>
<td>1.97</td>
<td>0.38</td>
<td>0.29</td>
<td>0.22</td>
<td>0.35</td>
<td>0.16</td>
<td>0.32</td>
<td>0.12</td>
<td>0.14</td>
</tr>
<tr>
<td>Nitrogen oxides</td>
<td>2.17</td>
<td>0.91</td>
<td>0.93</td>
<td>0.47</td>
<td>0.76</td>
<td>0.76</td>
<td>0.6</td>
<td>0.41</td>
<td>1.33</td>
</tr>
<tr>
<td>Fuel consumption</td>
<td>51.17</td>
<td>14.3</td>
<td>10.08</td>
<td>7.72</td>
<td>6.52</td>
<td>5.9</td>
<td>5.86</td>
<td>6.06</td>
<td>5.45</td>
</tr>
</tbody>
</table>

The ratio of pollution emission and fuel consumption before and after the widening project was calculated using equation 1:

\[
EF_{\text{overall}} = \frac{\sum_{i=1}^{4} (VKT_i \times EF_i)}{\sum_{i=1}^{4} VKT_i}
\]

In this equation, \(i\) represents various time intervals, \(EF\), and \(VKT\), represent the ratio of emission or fuel consumption and the percentage of the traveled distance in the speed interval of \(i\), respectively [7]. The required emission ratio of this equation derives from table 2. The percentage of the traveled distance in every speed interval is determined using the traffic sensors, a sample of which is demonstrated in figure 3. Based on the traffic volume data shown previously in Table 1, the daily distance traveled by the cars in the area covered by the traffic sensors was measured. Moreover, by using the emission rates and fuel consumption after the widening project (which is derived from equation 1), the average decrease of the pollutants and fuel consumption in each of the days in a week is measured. After averaging the results, annual decrease for the gasoline vehicles is also measured (Tables 3 and 4).
Table 3. Percentage of the decreased emission and average fuel consumption (gasoline cars)

<table>
<thead>
<tr>
<th></th>
<th>Carbon dioxide</th>
<th>Carbon monoxide</th>
<th>Unburned hydrocarbons</th>
<th>Nitrogen oxides</th>
<th>Gasoline</th>
<th>Particles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Decrease percentage</td>
<td>17.58</td>
<td>15.94</td>
<td>19.2</td>
<td>11.7</td>
<td>17.3</td>
<td>-6.29</td>
</tr>
</tbody>
</table>

Table 4. Average amount of pollutants and annual fuel consumption (gasoline cars)

<table>
<thead>
<tr>
<th></th>
<th>Carbon dioxide(Tons)</th>
<th>Carbon monoxide(Tons)</th>
<th>Unburned hydrocarbons(Tons)</th>
<th>Nitrogen oxides (tons)</th>
<th>Gasoline (mm)</th>
<th>Particles (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Annual decrease</td>
<td>1182</td>
<td>51</td>
<td>2</td>
<td>4</td>
<td>699</td>
<td>-2</td>
</tr>
</tbody>
</table>

According to the above table, the ratio of the gas emission, carbon dioxide and fuel consumption after the widening project has decreased while the particle emission experienced an increase. It is worth noting that because of the limitations of equipment and study resources, the measurements of the emission ratio and fuel consumption was only done for the cars with gasoline engines. Since direct measurement of emission rate and fuel consumption reduction was only possible for gasoline cars, for the rest of the fleet authors relied on numbers published in the official reports of the Tehran Air Quality company which is a subsidiary of Tehran Municipality. Table 5 shows a summary of the parameters. With the help of equation 2, average amount of annual pollutants (for all kinds of vehicles) is calculated and the results are shown in table 6. In equation 2, it is assumed that the process of pollutant production rate change and fuel consumption for all other types of vehicles in Tehran follows the same pattern.

Decreased amount of pollutant \( i \) or gasoline consumption concerning all vehicles=

\[
\text{Reducing the amount of pollutant } i \text{ or gasoline consumption concerning passenger cars} =
\]

The role of the passenger cars in pollutant \( i \) production caused by all kinds of vehicles

Table 5. The role of the passenger cars in pollutant emission and fuel consumption regarding to all kinds of vehicles

<table>
<thead>
<tr>
<th></th>
<th>Carbon dioxide</th>
<th>Carbon monoxide</th>
<th>Unburned hydrocarbons</th>
<th>Nitrogen oxides</th>
<th>Gasoline</th>
<th>Particles</th>
</tr>
</thead>
<tbody>
<tr>
<td>The role of passenger cars</td>
<td>55</td>
<td>45.11</td>
<td>55</td>
<td>32.11</td>
<td>55</td>
<td>1.82</td>
</tr>
</tbody>
</table>

Table 6. An average decrease of annual pollutant emission and fuel consumption regarding to all kinds of vehicles

<table>
<thead>
<tr>
<th></th>
<th>Carbon dioxide</th>
<th>Carbon monoxide</th>
<th>Unburned hydrocarbons</th>
<th>Nitrogen oxides</th>
<th>Gasoline</th>
<th>Particles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Annual decrease</td>
<td>2149</td>
<td>114</td>
<td>4</td>
<td>1139</td>
<td>1172</td>
<td>-112</td>
</tr>
</tbody>
</table>

As an example, figure 6 shows cumulative monthly gasoline savings for a one year period after the widening project.
By referring to nationwide annual fuel consumption statistics, and the subsidies allocated to gasoline, the costs imposed on government for each liter of gasoline was calculated. In a similar fashion, marginal cost of energy production in European Union was used, to calculate monetary value for reducing one gram of each pollutant. Table 7 shows the values in Iranian Rials (IRR).

Table 7. IRR value of reducing emission is each gram of pollutants and the decrease of gasoline consumption

<table>
<thead>
<tr>
<th>Pollutant</th>
<th>Value of pollutant reduction in IRR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon dioxide</td>
<td>3.024</td>
</tr>
<tr>
<td>Carbon monoxide</td>
<td>0.324</td>
</tr>
<tr>
<td>Unburned hydrocarbons</td>
<td>25.765</td>
</tr>
<tr>
<td>Nitrogen oxides</td>
<td>867</td>
</tr>
<tr>
<td>Gasoline</td>
<td>2886/7</td>
</tr>
<tr>
<td>Particles</td>
<td>41640</td>
</tr>
</tbody>
</table>

After combining results presented in tables 6 and 7, the value of pollutant reduction and fuel consumption (concerning all kinds of vehicles) is calculated to be 42,885,000 IRR/day, and 15,610,750,000 IRR annually.

5. Conclusions
   1. After bridge widening (only in the area under research), citizens enjoyed 4700 hour of user time saving per day. In addition, project resulted in 18% carbon dioxide emission, 16% carbon monoxide, 19% unburned hydrocarbons, 12% nitrogen oxide and 17% gasoline reductions, however the particle emission only decreased 6%.
   2. Average daily economical savings from pollutant decrease, fuel consumption savings, and travel time improvement is estimated to be 513,226,000 IRR.
   3. Pollutant emission rates and fuel consumption in speeds below 20 k/h have a significant increase (up to 10 times more than the emission ratio and fuel consumption related to speeds above 40k/h).
   4. In Star Khan Project, it was observed that easing the congestion even in a limited scope (1400 meters), can have noticeable effects in travel time, ratio of pollutant emission and the fuel consumption. It also shows that changes in highway speed patterns may have major implications in air pollution reduction in metropolitan areas.
   5. This research was motivated and driven by high quality field data and analytical models including (1) large amount of traffic data gathered from Bluetooth traffic sensors installed by Mana Rika Company, (2) mathematical model of emission and fuel consumption of cars in Tehran which is
powered by data acquired from portable emissions test devices, (3) models for estimating costs imposed by production of pollutants and fuel consumptions.

6. Although the data was specific to this case study, the methodology can be easily adopted to measure and analyze the impact of transportation policies and major traffic improvement projects in a macro scale which is an effective tool for policy makers to understand and communicate societal and economic benefits of the projects to the general public.

6. References

PAPER TITLE: The Application of Innovative Geometric Designs to address Planning, Funding and Land Use Challenges

TRACK: 6.4: Coordination Between Land-use and Transportation Planning

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KEYWORDS:
Synchronized Split Phase Intersections, mobility, Performance Analysis, TransCAD, Vissim

ABSTRACT:
The gap between urban growth and land use planning in many communities is creating mobility challenges due to the difficulties in coordinating transportation infrastructure needs and land use. Those challenges are compounded by the rapid population growth, poorly designed roadway networks and economic losses from high traffic congestion. Many of those challenges could be alleviated by improved connection and balancing between mobility needs and land use to create livable communities.

This paper presents a case in the city of Riyadh (KSA) where there is an increased recognition of the urban land use planning and the importance of travel demand models. Specifically, this paper focuses on a corridor that was selected to be upgraded to a principal arterial in the city to improve the connectivity and roadway network hierarchy in the city. A certain section of the corridor upgrade faced challenges related to funding and land use accessibility needs that limited the possibility of applying grade separation of the intersection on that corridor section. To solve that issue an unconventional geometric design solution called “Synchronized Split Phase Intersections” was applied at the site with great success. The results of the before and after study are presented in this paper along with the major issues that were encountered during the development of the design and performance analysis. The results showed improvements in the level of service for the intersections and reduced delays despite the increased demand on the corridor after the upgrade of the corridor.
The Application of Innovative Geometric Designs to address Planning, Funding and Land Use Challenges

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1.0 INTRODUCTION

The city of Riyadh (KSA) is considered to be one of the most important and fastest growing cities in Saudi Arabia and the region with an average annual growth rate of 8% (Garba 2004). Its population has grown from around 700,000 inhabitants in the 1970’s to more than 6.5 million in 2016 (The General Authority for Statistics, KSA) and it is expected to maintain a high growth rate for years to come.

The need to control and direct the urban growth in Riyadh City lead to the formulation of the first Master Plan for Riyadh in the early 1970’s (Al-Hathloul, S. 2017), this plan aimed to provide guidance for the growth of the city up to the year 2000. The drastic changes to the economic and social aspects of the urban development in Riyadh following the oil boom in the mid 1970’s emphasized the need to have a well-structured urban growth planning process that corresponds to the city’s rapid growth. This materialized into several contracts between the Saudi Government and the different consultants to update and develop the Master Plan for Riyadh City until the Arriyadh Development Authority (ADA) was established and took several strategic initiatives to help guide the city’s rapid urban growth and maintain the livelihood of the city (Al-Hathloul, S. 2017).

The unconventional urban growth rate in Riyadh resulted in some gaps between land use planning and the needed transportation infrastructure to support that growth. To help better estimate the impacts of changes in the land use and transportation infrastructure on the demand for travel in the city, the Municipality of Riyadh (Amana) developed and updated the travel demand model for Riyadh. This model is being used to evaluate the different future alternatives related to land use and transportation infrastructure in the city. In the following sections, this paper provides a detailed description of the metropolitan travel demand model for the city of Riyadh and its use. Also, this paper presents a case study of a corridor that was selected to be upgraded to a principal arterial in the city in order to improve the connectivity and roadway network hierarchy in the city. A certain section of the corridor upgrade faced challenges related to funding and land use accessibility needs that limited the possibility of applying grade separation of the intersection on that corridor section. To solve that issue an unconventional geometric design solution called “Synchronized Split Phase Intersections” was applied at the site with great success. The results of the before and after study are presented in this paper along with the major issues that were encountered during the development of the design and performance analysis. The results showed improvements in the level of service for the intersections and reduced delays despite the increased demand on the corridor after the upgrade of the corridor.

2.0 CASE STUDY

The Municipality of Riyadh developed and updated a travel demand model for the city to be used for the analysis of different future development scenarios in the city. Following is a description of the travel demand model and the case study used.

2.1 Riyadh Travel Demand Model

The City of Riyadh has an estimated 7.4 million daily trips on its roadway network travelling a total distance of 102 million kilometers, and almost 77% of those trips are made by personal vehicles as shown in Table (1) which reflects the official numbers provided by the Municipality of Riyadh. Like many of the other major cities in the world,
the city of Riyadh is suffering from deteriorating level of service (LOS) and increasing travel delays affecting the mobility needs of the roadway system users. A major cause of this is the gap between urban growth and land use planning which is creating mobility challenges due to the difficulties in coordinating transportation infrastructure needs and land use. Many of those challenges could be alleviated by improved connection and balancing between mobility needs and land use to create livable communities. In the city of Riyadh there is an increased recognition of the urban land use planning and the importance of travel demand models to provide better analysis and utilize the available resources to the best possible extent.

<table>
<thead>
<tr>
<th>Attribute</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jurisdictional area (km)</td>
<td>4,614</td>
</tr>
<tr>
<td>Developed area (km)</td>
<td>1,913</td>
</tr>
<tr>
<td>Population</td>
<td>6,506,700</td>
</tr>
<tr>
<td>Auto ownership (veh/HH)</td>
<td>1.8</td>
</tr>
<tr>
<td>Number of workers</td>
<td>2,283,073</td>
</tr>
<tr>
<td>Number of daily trips</td>
<td>7,400,000</td>
</tr>
<tr>
<td>Daily VKT (km)</td>
<td>102,000,000</td>
</tr>
<tr>
<td>Daily VHT (hours)</td>
<td>2,199,000</td>
</tr>
<tr>
<td>% of private vehicles trips</td>
<td>77</td>
</tr>
<tr>
<td>% of public transit trips</td>
<td>3</td>
</tr>
<tr>
<td>% of heavy vehicles trips</td>
<td>20</td>
</tr>
</tbody>
</table>

The city of Riyadh has 15 sub municipalities which include approximately 170 neighborhoods, and all of them are under the jurisdiction of the Municipality of Riyadh (Amana). The Amana developed and updated the Riyadh Travel Demand Model (RTM) to better estimate the impact of the socioeconomic and infrastructure changes to the users of the transportation system. The current RTM has more than 2450 traffic analysis zones (TAZ) with the different land uses and socioeconomic properties updated and coded. The RTM update included adding 1780 km of new roads and updating the characteristics of 940 km of the modeled roads from the previous model. The updated RTM also included updating the land use data for 400 TAZ’s and 18 super zones as shown in figure (1). It represents a significant effort by the Amana to have a reliable well-structured approach to evaluate the effects of the different changes on the land use and infrastructure in the city.

2.2 Prince Turki Bin Abdul Aziz Al Awwal Corridor Upgrade

The official in the city of Riyadh took the decision to upgrade several corridors in the city to provide a roadway network with proper functional classification that could provide the needed accessibility and mobility needs to the users of the network. There are several roadway upgrade projects planned in the city of Riyadh, table (2) provides a summary of the existing and planned different roadway class lengths in the city of Riyadh based on the projects that have been approved by the Amana officials.
Table (2): Exiting and future roadway lengths by functional class in the City of Riyadh

<table>
<thead>
<tr>
<th>Roadway functional class</th>
<th>2016 (existing) roadway length (km)</th>
<th>2025 (planned projects) roadway length (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Expressway</td>
<td>470</td>
<td>820</td>
</tr>
<tr>
<td>Principal Arterial</td>
<td>480</td>
<td>810</td>
</tr>
<tr>
<td>Minor Arterial</td>
<td>340</td>
<td>460</td>
</tr>
<tr>
<td>Collector</td>
<td>1,930</td>
<td>2,130</td>
</tr>
</tbody>
</table>

The case study presented in this paper is the “Prince Turki Bin Abdel Aziz Al Awwal” corridor which was selected to be upgraded into an expressway to ease the North-South movement of the vehicles in the northwestern part of the city as shown in figure (2).

Figure (2): Prince Turki Bin Abdul Aziz Al Awwal corridor

This corridor was selected to be upgraded to relieve some of the traffic congestion on the parallel Takhassosi Rd. and King Fahad Rd. The upgrade included introducing grade separated intersection by providing tunnels for the through traffic movement on Prince Turki Rd. on its intersections as shown in figure (3). The construction has already finished on some of the intersections and will finish very soon on the remaining intersections except for the intersection of Prince Turki and Mecca roads. This particular intersection faced many challenges that affected the planned upgrade to grade separated intersection and had to be treated in different way than the other intersections on the corridor.
2.3 The Intersection of Prince Turki Bin Abdul Aziz Al Awwal and Mecca Roads

The upgrade of the intersection between Prince Turki and Mecca roads faced many challenges related to funding, adjacent land use and accessibility needs. The intersection and surrounding land use is shown in figure (4). As can be seen from the figure, there is an underpass on Mecca Rd. and there was a plan to have another overpass on Prince Turki Rd. to improve the North-South traffic movement on the corridor. The surrounding land use included schools, residential buildings, embassy and several highly active commercial complexes. The addition of the overpass would have a severe impact on the accessibility to the surrounding land uses which already exist. Also, with the planned opening date for three of the grade separated intersections to the north of this location there were many fears that this intersection would have increased traffic volumes and severe delays due to the existing conditions at the site which will affect the success of the corridor upgrade and the resources that were invested in it.
Based on the previous discussion, efforts were made investigating other alternatives for the intersection configuration that could provide the needed levels of accessibility to the surrounding areas. Also, the suggested alternative shouldn’t involve major construction efforts to have it ready by the planned opening date of the other upgraded intersections to the north of the location. The RTM was used to estimate the future volumes on the corridor after the planned changes to the roadway network and those volumes were also used in the traffic analysis for the different alternatives using PTV Vissim software. The results of the analysis concluded that the best alternative to be applied at the location given the local conditions was a modified synchronized split phase intersection. The description of the design and the analysis are provided in the following sections.

3.0 MODIFIED SYNCHRONIZED SPLIT PHASE INTERSECTIONS

The synchronized split phase intersection is an unconventional geometric design that is used for surface intersections and could be applied under certain conditions with relative success when compared to the conventional geometric designs under the same conditions (FHWA 2010). This type of design is also known in some parts of the world as the double crossover intersection. The main concept from the design is to allow the through and left-turn movements on the mainline to cross over before the main intersection. For the main intersection traffic, the through and the opposing lefts can move concurrently during the same signal phase. This intersection can then operate with two phases. Figure (5) illustrates the main movements on synchronized split phase intersection.

This unconventional design is perceived to be most beneficial in improving the level of service (LOS) in locations where high left-turn and through volumes lead to high delays. The design enables the signal phases to be reduced by allowing movements from the ramps to proceed concurrently with the through movements on the crossroad. The result of this will be that signalized crossovers operate with two-phase signal control. Another main advantage to
this type of design is that it improves the traffic safety at the locations because it has fewer conflict points compared to a conventional design and this should reduce the number of crashes at the site.

However, one of the major issues which the designer should make every effort to address is the driver confusion that may result from the reversed direction of travel for drivers between the two edge intersections. This could be reduced by introducing proper curve designs to channelize the traffic, proper traffic signage and lane marking. Also, new innovations in intelligent transportation systems (ITS) could help guide drivers throughout the intersections area.

3.1 Access Management Considerations

Due to the special nature of the traffic movement on this type of design there should be special consideration to the effects of applying this design on the adjacent land uses on case-by-case basis. This type of design is usually applied within urban areas and because of the special nature of the design there is a need for the inclusion of frontage roads to provide the needed accessibility for the adjacent lands. This will create a need to include additional phasing into the signalized intersections in the design which will impact its efficiency. Another accessibility issue is the restrictions on the U-turn movements mainly due to the crossover of traffic movements within the design. If the road width is acceptable, signalized protected U-turn movements could be incorporated within the wide medians upstream and downstream from the main intersection. However, their signal phasing must be synchronized with the signal phasing of the other intersections to make sure there is no conflict between the different traffic movements on the intersections and there is a smooth flow of traffic through the intersections.

4.0 TRAFFIC ANALYSIS

The traffic analysis process included performing traffic counts and roadway inventory on the intersection to estimate the existing traffic operating conditions. That was followed by using the RTM to estimate the short term future traffic volumes that will be using the intersection area after the upgrade of the “Prince Turki” corridor.

Microsimulation was used to analyze the performance of the intersection area for the different scenarios using PTV Vissim Software. The results of the analysis showed that the synchronized split phase intersection design provided the required levels of operational performance and accessibility at the intersection site. The analysis showed that the application of this design will help reduce the congestion at the site by operating the intersection as a two-phase signal instead of the existing four-phase signal. The synchronized split phase intersection design also provided the needed levels of accessibility to the different land uses surrounding the intersection through the provision of adequate U-turn movements and service roads. The results of the delay and traffic volume comparisons between the existing condition and the suggested design from the RTM and Vissim are provided in Figure (6) and Figure (7) respectively. As for the Maximum queue length comparison, it is provided in Figure (8).

![Average Vehicle Delay (sec.)](image)

Figure (6): Vehicle average delay at the intersection for existing and suggested design conditions
As can be seen from the previous figures, the RTM showed there will be an increase in the volume of vehicles attracted to the intersection by 20% due to the upgrades planned for the Prince Turki corridor. However, due to the design features of the synchronized split phase intersection and the traffic volume of the different movements at the intersection the micro-simulation showed that there will be a reduction of the average vehicle delay by 130 seconds. This indicated that the LOS will change at the intersection from LOS “F” to LOS “B” without the need to go for grade separation at that location. The design proved to be the best solution at the site due to the heavy left turn traffic movements, and it was operated as a two-phase signal with minimal vehicle queues.

The design was constructed at the site at the end of the year 2016 and it has been operating with a great success. It is being monitored continuously by the city officials and efforts are being made to adapt it to other locations in the city of Riyadh after its great success. The final layout of the intersection is shown in Figure (9), and a photo of the intersection after it was constructed is shown in Figure (10).
5.0 CONCLUSION

Due to the gap between urban land-use planning and infrastructure needs planning there are many challenges facing effective transportation network upgrades. This paper presented a case in the city of Riyadh, KSA where a corridor upgrade was faced by financing, time and surrounding land-use issues that limited the application of traditional geometric solutions to traffic problems on the corridor. An unconventional geometric design solution called “Synchronized Split Phase Intersections” was applied at the site with great success, and the design proved to be the best solution at the site due to the heavy left turn traffic movements, and it was operated as a two-phase signal with minimal vehicle queues. This paper concluded that unconventional geometric solutions can be utilized effectively to balance
between mobility and accessibility levels at certain sites to deal with untraditional conditions that may exist as a result of the backlog in urban planning and exiting transportation infrastructure.

ACKNOWLEDGEMENTS
The main author was on a leave from the University of Jordan (UoJ) and working with SETS Intl. at the time and he would like to acknowledge the support received from SETS company and UoJ needed to complete this work. Also, special thanks go to all the team members who worked on this project with the Riyadh Municipality.

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Evaluation of Various Partial-Depth Repair Materials for Rigid Pavement

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**Keywords:**
Rigid pavement, patching repair, concrete material

**Abstract:**
One of the most disturbing and frustrating pavement distresses encountered by motorists is the potholes and spalls on concrete pavements. A priority for pavement engineers is to develop patching techniques, and to discover new quality materials to repair potholes and spalls on concrete pavements. The objective of this paper is to evaluate the performance of various types of patch materials for the repairs of potholes and spalls of concrete pavements. Laboratory compressive tests were first conducted on the composite specimens in order to determine the strength and bond of the patching material to concrete. Results were then used to determine which of the patch materials would be used in the performance testing. During testing, every distress is monitored closely by visual inspection for any signs of de-bonding near cracks, or a complete failure. The sum of load repetitions endured on the test track was used to equate the simulated life expectancy (SLE) of the materials. Recommendation and guidelines for the repairs of patching and spalls were established.
Evaluation of Various Partial-Depth Repair Materials for Rigid Pavement

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1 INTRODUCTION

There is a trend of growing traffic on interstate and urban highway systems today. The heavily traveled urban sections have begun to show effects of far exceeded traffic volumes, and environmental problems. Many Portland cement concrete (PCC) pavements are getting older and showing rapid deterioration. Cracks, spalls, and other distresses are occurring on most roadways. To maintain the serviceability of the highway systems, materials with advanced technology must be applied to repair these distresses.

Pothole and spall repair can be improved by using quality materials in conjunction with proper patching procedures. Traditional repair techniques of potholes and spalls required crews to square-cut the edges of the distress, which increase risk damaging of the original pavement in the vicinity of the repair. Extremely high intensity vibrations caused by the diamond blade saw may propagate cracks along the pavement. In addition, a 90-degree corner is very difficult for road crews, and over extending a saw cut on the corners occurs frequently. This leads to the initiation of cracking in the future. However, with new high strength patching materials the straight edge cutting of concrete may not be necessary. New concepts are needed to incorporate and apply recent advances in material technology to improve the performance and service life of highway pavements. Industry has produced many high-strength, fast-cure patching materials to enhance concrete pavement performance. Along with these materials, companies have been able to keep mixing and application procedures simplified. These fast set materials are able to minimize curing time allowing for less traffic congestion.

The objective of this research study was to evaluate the performance of patch materials for partial depth repairs of potholes and spalls from concrete pavements.

2 BACKGROUND

2.1 Partial-Depth Repairs

The American Concrete Pavement Association (ACPA) (1) reported on the guidelines for partial-depth repair. Partial-depth patches are placed to repair spalls either at pavement joints or at mid-slab locations. Spalls create a rough ride and can accelerate further deterioration. Spalling is a localized distress, and therefore, warrants a localized repair procedure for the pavement to be restored. Repair of this distress is needed to improve rideability, deter further deterioration, and provide proper edges so that the joints and cracks can be resealed effectively. It was learned that a good performance of partial-depth repairs can be obtained by:

- Limiting use of the technique to the top one-third of the slab and not extending repairs to a depth that allows the patching material to bear directly on dowel bars or reinforcing steel.
- Inserting a compressible material in all working joints and cracks or adjacent to the patch. The compressible material should extend 25.4 mm (1 in) below and 76.2 mm (3 in) laterally beyond patch boundaries.
- Using a bonding agent compatible with the selected patching material. Incompatibility will likely result in delaminating.
- Sealing the patch/slab perimeter interface using cement: water grout for cementitious patch materials to prevent moisture infiltration.
- Resealing the joint after repair to prevent water and incompressible from causing further damage.

Darter et. al. (2) reported on the design guidelines of partial-depth patches. It was stated that the type of patch material to use for partial-depth patching depended on such factors as amount of time before opening to traffic, ambient...
temperature, cost, size, and depth of patches. The success of partial-depth patching depends on an adequate bond to existing concrete, therefore it is important that proper surface preparation to the concrete be done.

2.2 Performance of Patching Materials

Peshkin (3), as part of the Strategic Highway Research Program's (SHRP) initiative in highway operations area, studied the effectiveness, equipment, and procedures of partial-depth spall repair in Portland cement concrete (PCC) pavements. Test sites for the research were installed in highways across the United States and Canada. Installation and performance data was compiled and analyzed to provide preliminary indications about distress development and survival rates of various repairs. Under this project, 1,600 spalls were constructed with the cooperation of 15 different state DOT's, 1 Canadian Province, and 1 city Department of Public Works. Private contractors installed two of the sites; while the sponsoring agency performed the rest of the work. Laboratory tests were performed on the repair materials and the data were used to identify correlation between laboratory test results and field performance. The research concluded that agencies might be able to save significant portions of their maintenance budgets and greatly increase the effectiveness of their repair activities by using higher quality materials.

Parker and Shoemaker (4) conducted studies on laboratory and field performance of three rapid-strength PCC pavement patch materials. The three selected materials were a rapid-setting PCC mixture, a rapid-setting fibrous PCC mixture, and Road Patch II, a proprietary material. The fibrous PCC contained discrete steel fibers. Laboratory mix designs revealed that PCC with and without steel fibers and the Road Patch II could produce an early compressive strength adequate for one-day patch construction. Laboratory tests showed that four-hour compressive strength tests of the PCC materials were higher than the proprietary material.

Ramey et. al. (5) reported on the strength and weathering characteristics of selected rapid-setting PCC pavement patching compounds. A laboratory testing program evaluated material and bonding properties, which are fundamentally related to the durability and performance of spall-type patches. Patching materials used for shallow-depth surface repairs of PCC pavements slabs and bridge decks were selected. Polymer concrete, Magnesium Phosphate Cement (MPC), Road Patch with steel fibers, and Epoxy/PCC were the four rapid setting materials chosen. Testing procedures included; compressive strength, tensile strength, direct shear, and impact tests. Three test series were employed to evaluate the effect of age and simulated weathering exposure.

3 LABORATORY TESTING PLAN

A total of sixteen composite specimens from the donated materials were cast in cylinders for compressive testing. The purpose of this test is to simulate conditions patching materials may exhibit during pavement rehabilitation. Various composite configurations are shown in Figure 1. Due to limited materials supplied by the manufacturers and the difficulty in casting the composite specimens, it was decided to only cast 1 or 2 cylinders per configuration. A simple technique for casting composite specimens may need to be studied in the future. The cylinders were standard 152.4 mm x 304.8 mm (6 in x 12 in) size. Type I PCC was used for the concrete part. The concrete cylinders were allowed to cure for 28 days in water baths before application of the patching materials. Thereafter, the five patching materials were prepared and arbitrarily placed in the ready-formed cylinder. The patching materials were alphabetically labeled on all composite specimens. Materials D, E, and G were elastomeric concrete, while materials F and H were a cementitious mix. Although duplicate configurations were made, only one cylinder was cast with each composite material.

Results of the compressive strength tests are shown in Table 1. Good bonding and high compressive strength of the patching materials in composite specimens were the two criteria for selecting the materials to be tested for performance. Table 1 also shows the average compressive strength to be 48.5 MPa (7,034 psi) for the control sections. It is imperative to say that for the composite cylinders, the aspect ratio (LID) of the concrete part will not be equal to two as is for the ASTM standard cylinders. Therefore, the compressive strength for the composite specimens will be very difficult to predict. All patching configurations (PAC) numbers were taken from Figure 1. Materials D and G were both tested from PAC 1. The compressive strengths of material D and G only reached 7.32 MPa (1,061 psi) and 4.88 MPa (707 psi) respectively because of the early failure at the interface. However, composite cylinder G shows the cracks occurring at the interface and through the PCC under the compressive test. For this specimen, it was obvious that the patching material could sustain a higher load than the PCC but was still weak in bonding.
Figure 1. Composite Specimen Patching Configurations
### Table 1. Laboratory Compressive Test Summary Results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Config. No.</th>
<th>Load kN (lbs)</th>
<th>Strength Mpa (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control Specimen</td>
<td>874 (196,500)</td>
<td>47.9 (6,950)</td>
<td></td>
</tr>
<tr>
<td>Type I Cement; 28-day cure</td>
<td>912 (205,000)</td>
<td>50.0 (7,250)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>867 (195,000)</td>
<td>47.6 (6,897)</td>
<td></td>
</tr>
<tr>
<td>Control Specimen</td>
<td>945 (212,000)</td>
<td>51.7 (7,498)</td>
<td></td>
</tr>
<tr>
<td>Type II Cement; 1-day cure</td>
<td>923 (207,500)</td>
<td>50.6 (7,339)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>930 (209,000)</td>
<td>51.0 (7,397)</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>133 (30,000)</td>
<td>7.32 (1,061)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>191 (43,000)</td>
<td>10.5 (1,521)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>189 (42,500)</td>
<td>10.4 (1,503)</td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>534 (120,000)</td>
<td>29.3 (4,244)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>478 (107,500)</td>
<td>26.2 (3,802)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>334 (75,000)</td>
<td>18.3 (2,653)</td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>156 (35,000)</td>
<td>8.53 (1,238)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>698 (157,000)</td>
<td>38.3 (5,553)</td>
<td></td>
</tr>
<tr>
<td>G</td>
<td>89 (20,000)</td>
<td>4.88 (707)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>300 (67,500)</td>
<td>16.5 (2,387)</td>
<td></td>
</tr>
<tr>
<td>H</td>
<td>423 (95,000)</td>
<td>23.2 (3,360)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>778 (175,000)</td>
<td>42.7 (6,189)</td>
<td></td>
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<tr>
<td></td>
<td>756 (170,000)</td>
<td>41.5 (6,013)</td>
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</tr>
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</table>

Material D also tested on PAC 5 and 8 shows an interesting fracture of the concrete, but the concrete portion surrounding the patching material remained in its cylinder shape. The compressive strength of both specimens only reached approximately 103 MPa (1,500 psi). This may be due to the higher aspect ratio of PCC while the patching material was confined by the concrete. According to Mohr's failure theory, the material can sustain a higher load with application of a confining pressure. This may be the reason why the concrete failed before the patching materials. Three other composite samples were tested with material E from PAC's 4, 6, and 8. The compressive strength of composite cylinders was 29.3 MPa (4,244 psi), 26.2 (3,802 psi), and 18.3 MPa (2,653 psi) respectively. The fractures took place along the composite specimen. Since PAC 4, 6, and 8 were partially filled with patching material E, no de-bonding was permitted. A confining pressure contributed by the surrounding concrete may have increased the failure strength of the patching materials. Therefore the fracture would take place in the PCC and not in the patching material. PAC 4, 6, and 8 demonstrated this actually happening. The test further hinted that the high strength patching material could simply fill a typical shaped pothole without worrying about de-bonding.

PAC 2 and 6 cylinders were cast with material F. The compressive strength was 8.53 MPa (1,238 psi) and 38.3 MPa (5,553 psi) respectively. PAC 2 clearly failed in bonding, which resulted in a low compressive strength. The entire specimen fractured through patching material F and the PCC. This occurrence is evident because of the higher compressive strength of 38.3 MPa (5,553 psi). This material certainly would not be chosen for the accelerated performance test due to its poor bonding characteristics although it had a relatively high compressive strength. Material G was tested with PAC 1 and 3 cylinders. PAC 1 failed in bonding quickly at a very low compressive strength. For PAC 3, the aspect ratio was approximately 1 for the PCC. However, the compressive strength only reached 16.5 MPa (2,387 psi) when the concrete cracked. Due to this result, it is recommended that more samples be cast and tested in the future. Three composite specimens were cast with material H, in PAC's 4, 6, and 7 cylinders for compressive tests. The compressive strength reached approximately 41.4 MPa (6,000 psi) for PAC's 6 and 7, closely achieving the compressive strength of the control sections. However, PAC 4 only strength may be attributed to the conical shaped composite specimen, which allowed the higher strength patching material due to the confining pressure to wedge into the lower strength PCC causing failure in the PCC. Photographs 15 and 16 show the interesting fracture patterns of the laboratory samples. The final compressive tests were conducted on the fast-set Type II cement mix. As previously mentioned the Type II cement mix was used for the control section during the accelerated performance testing the two cylindrical specimens were allowed to cure for only 24-hours. Yet, the average compressive strength for the Type II cement mix was 51.2 MPa (7,419 psi)

### 4 TEST FACILITIES

The facility (7) comprises a test track 15.2m (50ft) in diameter, a variable weight-loading apparatus, and a power source. The current track surface is a 1.8m (6ft) width concrete pavement supported on an earth embankment. The loading system consists of three supports of 7.6m (25ft) long W36x150 steel beams spiked from a pivot at 120-degree intervals. Each support beam is attached to a hydraulically driven dual-wheel truck-axle assembly. A water tank 3.7m (12ft) in diameter by 2.4m (8ft) height is centrally mounted on top of the support beams and is used to create
additional weight to the loading system. The total weight of the loading apparatus and water can vary between 134kN (30,000lbs) and 356kN (80,000lbs) and is evenly distributed to the three dual-wheel assemblies. The entire loading machine is powered by a 220hp diesel engine with a hydraulic transmission and is capable of driving up to 48km/hr (30mph) speed in either clockwise or counter-clockwise direction. A center support assembly, used to hold the entire system in place, is designed to restrain the testing machine from horizontal movement while allowing free rotation, vertical movement, and a small amount of tilt. The facility used to simulate actual traffic loads applied to all tests, was a dual wheel loading of 44.5 kN (10,000 lbs). During testing, every distress is monitored closely by visual inspection to detect if any signs of de-bonding, wear, cracks, or complete failure have occurred. The sum of load repetitions endured on the concrete pavement will then be used to equate the simulated life expectancy (SLE) of the materials.

4.1 Pothole Construction on the Test Track

Figure 2 illustrates the test track with pothole layouts. Half of the track was employed to test the performance of patching materials for pothole distresses. It was proposed to have fourteen such potholes to be used in the accelerated performance test. Three different pothole sizes were created: seven 304.8 cm x 30.48 cm x 12.7 cm (1 ft x 1 ft x 5 in) (Detail 1), seven 60.96 cm x 30.48 cm x 12.7 cm (2 ft x 1 ft x 5 in) (Detail 2), and two feathered edged 304.8 cm x 127 cm (1 ft x 1 ft x 5 in) (Detail 3). As stated earlier, the other half of the test track was still being tested from a previous project. The concrete pavement was constructed to a 25.4 cm (10 in) in thickness for artificial pothole creation.

4.2 Application of Patch Materials

The placement of the patch materials onto the pre-formed potholes was not an easy task for this project. The manufacturer's instructions for placing the patching material were read carefully, noting all necessary materials and equipment to be used. The material had a flush finish with the vertical edge of pavement at the joint. The patching material was placed immediately after mixing and it was placed at once instead of in stages. The patch was finished from the center of the patch out to the edges so to maximize bonding capacity between the patch and the existing pavement, especially near the top of the patch.

Levelling the materials with the track surface became somewhat difficult with the cementitious materials and the control section. The addition of 6.8 kg (15 lb) to 11.3 kg (25 lb) of aggregate to the cementitious materials made for a fairly dry mix. The mix design for the fast-setting Type II cement required a 67-stone aggregate, thus the control section with became particularly difficult to place and finish properly. The volume of the aggregate also became a hindrance when levelling the patching material to the surface of the test track. Characteristics of the patching materials are summarized in Table 2.

<table>
<thead>
<tr>
<th>Material</th>
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<th>Workability</th>
<th>Curing time (hr)</th>
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<tr>
<td>A</td>
<td>Polymer</td>
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<td>Moderate</td>
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<tr>
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<td>Easy</td>
<td>Moderate</td>
<td>2</td>
</tr>
<tr>
<td>C</td>
<td>Cementitious</td>
<td>Easy</td>
<td>Moderate</td>
<td>4</td>
</tr>
<tr>
<td>D</td>
<td>Elastomeric</td>
<td>Easy</td>
<td>Easy</td>
<td>4</td>
</tr>
<tr>
<td>E</td>
<td>Elastomeric</td>
<td>Moderate</td>
<td>Moderate</td>
<td>4</td>
</tr>
<tr>
<td>H</td>
<td>Cementitious</td>
<td>Easy</td>
<td>Moderate</td>
<td>2</td>
</tr>
<tr>
<td>Control Section</td>
<td>Type II (PCC)</td>
<td>hard</td>
<td>Difficult</td>
<td>24</td>
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</table>

5 LIFE EXPECTANCY AND TEST OBSERVATIONS

A total of 500,000 load repetitions was applied. Although this quantity of repetitions is considered low, noticeable fatigue to some materials is present. Some stresses were observed in two materials. The 500,000 repetition represented a simulated life expectancy of 17.7 years, assuming an ADT of 10,000 with an average percent of heavy trucks of 6%.

Material D, and E, which are Elastomeric Concrete, was failing due to de-bonding. It was noticed that after placing material D, shrinkage quickly occurred in the morning showing cracks along the interface of the material and concrete slab. However, the separations due to de-bonding were not seen during the warmer times of the day. In case of Material E, de-bonding failure has occurred due to the fatigue testing. Shrinkage/expansion cracks along Material E’s surface were present. This was due to excessive addition of water while mixing the cementitious material. This factor was evident, because the other pothole with Material E present had no cracks or other failures associated with bad mixing procedures.
6 CONCLUSIONS

The results of the compressive strength tests indicated the configurations used to cast the composite specimens might be reliable in choosing the most dependable materials. All of the samples tested in the laboratory showed unique fracture planes and bonding characteristics with the concrete. Failures occurred between 7.32 MPa (1,061 psi) and 42.7 MPa (6,189 psi) depending upon the material, the patching configuration and the aspect ratio (L/D). The results of the fracture patterns and compressive strength data have been evaluated and conceptually analyzed. Many types of quality materials exist in today's market, and to select the most appropriate specifications and guidelines must be made for all manufacturers' to follow. The UCF-CATT facility was used to simulate actual traffic loads applied to all tests, was a dual wheel loading of 44.5 kN (10,000 lbs). During testing, every distress is monitored closely by visual inspection to detect if any signs of de-bonding, wear, cracks, or complete failure have occurred. The sum of load repetitions endured on the concrete pavement will then be used to equate the simulated life expectancy (SLE) of the materials. After a total of 500,000 load repetitions of 44.5 kN (10,000 lbs) wheel load applied at this test facility, some signs of patching...
distress have been found. The patches were observed and inspected daily, thus knowing the time any damage had incurred. At the end of the testing schedule, the 500,000 repetition represented a simulated life expectancy of 17.7 years, assuming an ADT of 10,000 with an average percent of heavy trucks of 6%. Some de-bonding failures have occurred. These failures occurred because of shrinkage due to weather changes, and de-lamination of the patching material from the concrete interface. Elastomeric and polymer materials particularly had shrinkage problems. Cracking also occurred in a cementitious mix due to excessive water in the mix design. However, failure from severe wearing, cracking, and spalling has not been observed of any of the patch materials. The two feather-edged potholes, which are simulating realistic pothole conditions on the highway, have performed well to this point. This has proven to be valuable, because if crews do not have to square-cut partial-depth patches, placing the material directly into the distress can save time and money. However, in determining the pavement life extension, both the life of the individual repair and the life of the pavement as a whole shall be accounted for. The life-cycle cost analysis will be based upon application of the patching material on an actual state highway requiring maintenance, or one with available maintenance data. The costs of each material will be determined and evaluated with its service life. This evaluation will then be compared to the present materials used in the rehabilitation of concrete pavements. The life-cycle cost analysis for each method can then be studied to determine the best alternative for partial-depth patching. This investigation is an important step in convincing engineers and agencies that there are many reliable materials for the use in today's overly traveled interstate and highway systems.

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